DEPARTMENT OF THE INTERIOR

UNITED STATES GEOLOGICAL SURVEY

GEORGE OTIS SMITH, Director

INSTRUCTIONS TO TOPOGRAPHERS

OF THE

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WASHINGTON

GOVERNMENT PRINTING OFFICE

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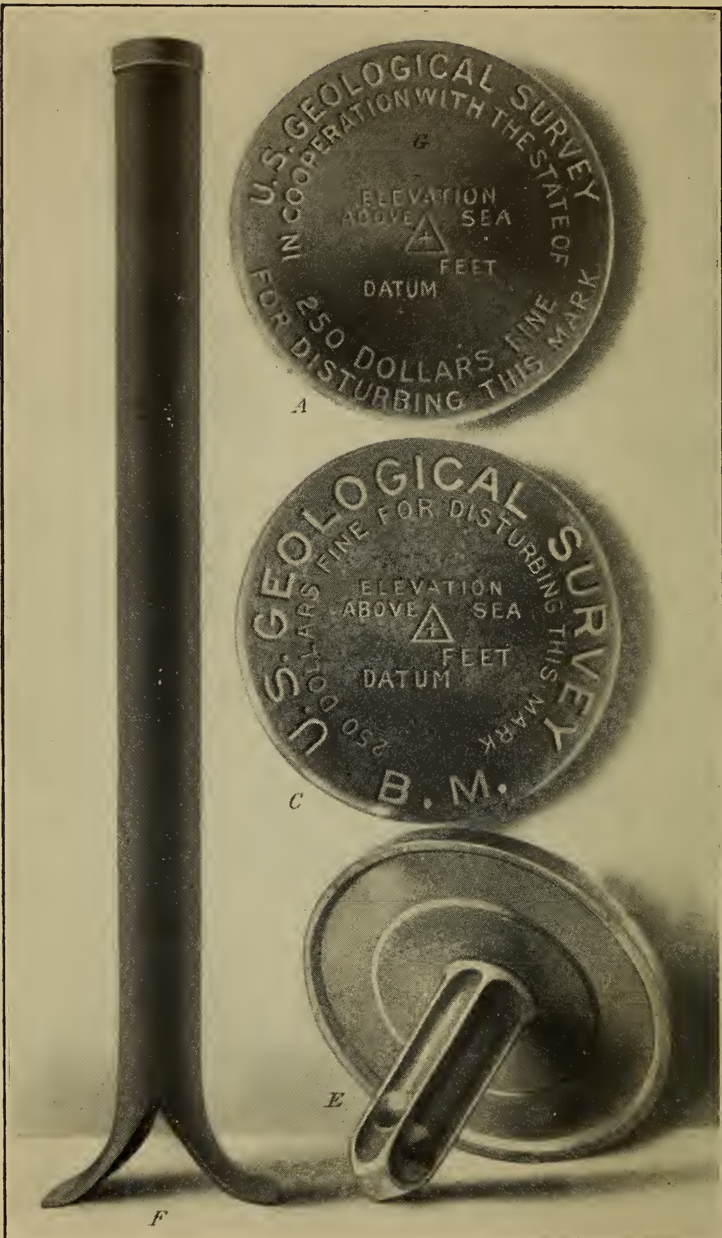
CONTENTS.

PREFATORY NOTE.

The following instructions will eventually be incorporated in a handbook of instructions which is in preparation for topographers of the Geological Survey. They are printed in the present form for temporary use, and any criticisms or suggestions that would tend to improve them should be sent promptly to the chief geographer.

R. B. Marshall,
Chief Geographer.

Approved:
Geo. Otis Smith, *Director.*
Washington, D. C., June 30, 1911.



MARKS FOR HORIZONTAL OR VERTICAL CONTROL STATIONS.

A, Tablet used in cooperating States. The State name is inserted at *G*.
A, *C*, and *E*, Tablets for stone or concrete structures.
F, Iron post used where there is no rock.

INSTRUCTIONS TO TOPOGRAPHERS OF THE UNITED STATES GEOLOGICAL SURVEY.

HORIZONTAL CONTROL.

GENERAL CONDITIONS.

The boundary lines of all regular United States Geological Survey maps are parallels of latitude and meridians of longitude. In order that these shall be properly located and that intermediate points shall be placed in correct positions according to scale, some system of horizontal control is required. The method to be adopted for linear control should be fixed by the character of the country, the requirement being that all control work shall be so accurate that no errors will be apparent in maps several times as large as those to be published. In mountainous regions or in hilly, partly timbered areas horizontal control is effected by a system of triangulation, the whole area being divided up into triangles whose apexes are represented by stations established on prominent points several miles apart. The angles between each station and all others visible from it are carefully measured with theodolites arranged to read angles as small as one second. One side of one of the triangles, called the base line, must be measured with great care with steel tapes, account being taken of slope of the line, elevation above sea, temperature of the tape, and other essential details, and for at least one station the exact latitude and longitude must be determined by astronomic observations.

In heavily timbered areas where it is difficult to see from any point more than a mile or two in any direction, horizontal control is best

obtained from distances actually measured on the ground with 300-foot steel tapes and angles measured with a transit at each bend in the line. Such control, called primary traverse, must begin and end at points whose positions have been previously determined, and be carried around the edge of each quadrangle and once across its center east and west.

Because of the great expense involved in base-line measurements and the fixing of astronomic positions, it is generally necessary to connect triangulation systems or traverse lines with positions previously determined, even though they may be some distance away. There are now but few localities in the United States that can not conveniently be connected with known positions and distances, and therefore, before horizontal control work is begun, the records of the Coast and Geodetic Survey, the Lake Survey, the United States Army Engineers, and other Government organizations should be examined in order to ascertain what positions in the area surveyed have been determined and are available for use in the work on hand.

The results of triangulation or primary traverse by the Geological Survey can always be obtained by anyone having occasion to use them by applying to the Director, United States Geological Survey, Washington, D. C.

PRIMARY TRIANGULATION.

FIELD WORK.

On the flyleaf of each field notebook is a blank in which shall be recorded all information necessary to identify the book. This blank should be filled so far as practicable on or before the first date of entry of field notes, and it must be completely filled before the book is forwarded to the Washington office. Any failure to fill in completely the blank on the flyleaf of a field notebook should be reported by the computer to the geographer in charge of the division.

Personnel of party.—Each party usually consists of a triangulator and a recorder; also a cook and a teamster (or packer) in regions where camping is necessary. Additional men are required for heliotroping, one for each heliotrope station, and local laborers may be employed to clear timbered summits or to erect large signals.

Instruments, tools, books, etc.—The following instruments and books are used in primary triangulation:

One 8-inch theodolite, with leather carrying case and shoulder straps.
Two pairs field glasses.
One prismatic compass.
One protractor (6-inch celluloid, full circle).
One boxwood scale, graduated to inches and tenths.
One 50-foot steel tape.
One electric hand lamp.
One 6-foot steel tape.
Heliotropes.
One plumb bob.
Triangulation tablets or posts, according to requirements of country.
Cement, cans.
Signal notices, printed on cloth.
Climbing irons, for use in wooded regions.
Sun umbrella } For use in regions where improvised sun and wind shelters
Wind screen } can not readily be built.
Triangulation field notes (9–912).
Computation of geodetic distances (9–901).
Computation of geodetic coordinates (9–902).
Computation book, blank (9–989).
Nautical almanac (abridged).
Geographic tables and formulas.
Seven-place logarithm tables.
A good watch must be provided by the chief of party.

The following additional articles may be purchased in the field: Ax, hatchet, saw, nails, tacks, signal cloth, guy wire, stone drills (1⅛-inch bit), drill hammer, post-hole digger, wire cutter, brace and bits.

Amount of control.—At least three serviceable stations must be established in each quadrangle and as many more as may be necessary to afford adequate control. In addition, a number of secondary points—such as church spires, windmills, water tanks, trees, and in high mountain regions some of the more prominent summits—must be located by intersection or by the "three-point method." Where no such objects are available, at least two points should be flagged for intersection if practicable. These points are intended to afford supplementary control for the topographer and should be selected with special reference to their usefulness in that connection.

The triangulator is also expected to locate, when practicable, either by direct measurement from his stations or by the three-point method, conspicuous objects, marks on State and county boundary lines, and township and section corners. Especial attention should be given to township and section corners because of their recognized value in the control of the land-line net.

Reconnaissance.—Stations should be selected and signals built before any observing is done, and to this end the triangulator and his assistant should make a reconnaissance over the area to be controlled. Such reconnaissance should disclose every practical scheme of triangulation, the angles at each point selected being measured with a prismatic compass and platted with the protractor so that the size and proportions of the figures may be ascertained. All preparatory work, such as the setting of tablets and posts, the erecting of signals and scaffolds, and the clearing of lines of sight, should be completed during this reconnaissance, so that the final observing may be performed with economy and dispatch. The reconnaissance affords the triangulator opportunity to acquaint himself with the shortest routes of travel, with the best stopping places, with the available camp sites, water holes, pastures, and trails, and with the best routes for scaling each peak to be occupied, and it enables him to gain a familiarity with the special character of each station and its signal which will be invaluable to him in identifying the points when he sights them later on.

Figures.—The most desirable groups of triangles consist of either quadrilaterals with both diagonals sighted or central point figures with four to seven sides. The triangles composing these figures should be well proportioned, angles measuring not less than 30° nor more than 120° each. The scheme should not be allowed to dwindle down to simple, unsupported triangles, and especial care should be taken to connect the work done with other work by means of well-proportioned triangles. Overlapping figures or an excess of observed lines beyond those necessary to insure a double determination of each length are undesirable, although an occasional diagonal through some figure may be valuable as a check. Additional lines of this kind only complicate the main scheme without materially adding to its strength, and the numerous observa-

tions made for them are discarded by the computers as superfluous. Judgment is to be used in this matter, however, for in many regions the atmospheric conditions are exceedingly uncertain and the topographer can not always count on being able to observe in both directions over every line that may be essential to the main scheme. In such regions it is well to err on the safe side and to obtain too many data rather than too few.

Angles should be read to all prominent points outside of the area for use in future expansion, even though they are without signals or are not sharply defined.

Secondary points.—In cutting in secondary points for topographic control it should be remembered that locations which depend on two sights only, even if the angles are of adequate size, are likely to be of doubtful value, because of the absence of any check on possible gross errors in observing or computing, or because of mistakes in the identification of the points. An endeavor should therefore be made to obtain at least three sights to every secondary point, even if the triangles are not of the best shape. Triangulators are especially cautioned not to slight the location of secondary points merely because they happen to be of no importance in their scheme of figures. The topographer may find it expedient to start his control from a secondary point, so that a blunder in the location of such a point may result in his starting with an erroneous base and having to make corrections at a great cost.

Consent of owner.—Before a site for a station on private land is selected, the written consent of the owner should be obtained, if practicable, for establishing a permanent station mark and erecting the required signal. If a summit must be cleared of timber, or if lines of sight must be cut, the value of the timber to be cut should be definitely fixed and agreed upon with the owner before cutting is begun. Payments on this account should be made and sub-vouchers taken before the station is left. A suggested form for wording these vouchers is as follows:

Received from .... the sum of .... in full payment for all damages incident to destruction of timber on .... hill (or mountain), in .... County, State of ...., in connection with the establishment and occupation of triangulation station .... [give name of station], ... [date].

When it is necessary to clear away timber and the owner or agent for the ground can not be reached without great delay, three residents of the locality should be asked to appraise the value of the timber cut and to sign a written statement regarding it. This statement should be forwarded to the office of the Survey for consideration should a claim for damage be filed.

Station marks.—Primary triangulation stations must be permanently marked by either standard iron bench-mark posts or by tablets, each tablet to be set in rock in place or in the top of a concrete or stone monument. (See second paragraph, p. 53, for instructions regarding the setting of tablets.) When practicable, bottles or other imperishable material should be left as a subsurface mark.

Two or more permanent reference marks should be established about each station mark. They may consist of holes drilled in rock in place, spikes in roots of trees, or large stones set solidly in the ground. The azimuth and the distance to each reference mark must be duly entered in the field record.

When old stations are revisited and any of the marks are found to be defective or to have been destroyed, new marks must be established in their place.

Signals.—Triangulation signals must be built with a view to their permanence as well as to their visibility. They may be of various forms, the form selected depending on the locality and the materials at hand. Thus, a signal on a bare mountain peak may be a rock cairn; one on a partly wooded summit may be a straight tree, the surrounding timber being cleared away; one on cleared land may be a tripod or quadripod.

Rock cairns should be not less than 8 feet high and should be well put together, so that they will withstand strong winds and heavy snows. A pole or a small green tree placed in the top is of advantage in sighting.

Signal trees are most satisfactory if stripped of their branches, except a tuft at the top. They form the best of targets when sighted against the sky, but if they are to be sighted against a dark background they should carry two triangular targets 3 to 6 feet on a side, placed at right angles to each other and covered with white cloth. Tripods or quadripods should be built of sawed lumber if such

material is available. For the legs and center pole 2-inch by 4-inch scantlings may be used, for the cross braces 1-inch by 6-inch boards. The base of the pyramid should be large enough to permit a theodolite to be set up under the center pole. In order to increase its visibility, boards may be nailed across the sides about a foot apart and covered with signal cloth, and cross targets may be attached to the center pole above the apex of the pyramid. The best colors for this cloth are white and black or white and red.

Most signals stand in exposed places and should be securely anchored to prevent their being blown over. The legs of tripods and quadripods should be planted in the ground at least 2 feet; each should be fastened to a "deadman" and the holes filled with thoroughly tamped earth or rocks, or else a stake 4 feet long should be driven into the ground at an angle with each leg and firmly spiked to it. If the ground is too rocky to permit the digging of holes, a 4-foot crosspiece should be nailed to each leg at right angles, flat on the ground, and weighted down with rocks.

Scaffolds.—If it becomes necessary to elevate the instrument a scaffold must be erected in the form of a tripod, capped with a thick board 12 inches square, to support the instrument. Around this scaffold, entirely independent of it, should be built another, in quadripod form, supporting a platform on which the observer is to stand. If very high, such a scaffold should be composed of successive bents, each 8 or 12 feet, with diagonal bracing. The outer scaffold, further, is to serve as a signal, and for that purpose should extend at least 6 feet above the observing platform and be surmounted by a mast bearing cross targets. Before fixing signals in position the direction in which sights are to be taken should be carefully ascertained, so that no woodwork will interfere with the observations.

The size of the timbers to be used necessarily depends on the height of the structure. The amount of lumber required may be determined by means of a rough drawing of the structure to scale.

Centering of signals.—Great care must be taken to insure perfect centering of signal and scaffold over the station mark, the plumb bob being used for this purpose. Signals should stand over station marks wherever possible, so as to avoid the necessity of computing

swings for the angles, but if this is impracticable, as it is with a tree signal, then the distance and bearing of the signal to the station mark must be carefully measured and recorded.

The permanent mark, tablet, or post must be the station, and when observations are made for angles the theodolite should be set up over its center if possible. If it is impracticable to center the instrument over the station mark the distance between the point occupied and the station mark must be carefully measured and recorded. Also one or more sets of angles must be read between the station mark and the other stations, in order of azimuth, preferably with the 0° 0′ for the pointing to the station mark.

Heliotroping.—The heliotrope outfit commonly used by the Survey is either the Steinheil heliotrope or a plane mirror with a screw hinged to the back to give it universal motion and improvised diaphragms of tin or wood with round apertures. The plane mirror is generally preferred to a heliotrope of the more elaborate form.

A heliotrope is usually set up by mounting the mirror on a stake or board immediately over the center of the station and the diaphragm on another stake, 10 or 20 feet away, which is carefully lined in with the distant station. The operator must constantly watch the reflected image of the mirror and keep it symmetrically over the aperture. If the sun is back of the observer a second mirror placed at a distance of a foot or two from the first may be used to reflect the light into the first.

To the observer the flash should appear as a clearly defined point of light; if of appreciable size it will be necessary to bisect it, and an error is thus likely to be introduced. A good rule to follow is to make the diameter of the opening in inches equal to one-fiftieth of the distance in miles for work in the West, and twice this size for work in the East, with a minimum opening of one-quarter inch.

Time of observing.—As a rule the best time for observing is the three hours before sunset; the atmosphere is then steadiest and shows no "boiling." The early morning hours are occasionally good but are likely to be less satisfactory. Many cloudy or overcast days are favorable. As a last resort observations at night may sometimes be necessary, but these require special night signals and assistants to

operate them, and because of the additional cost involved are seldom warranted.

Preparation for observing.—Whenever practicable the theodolite must be set over the station mark for reading angles, to obviate reduction to center. In setting up the tripod the head bolt thumb-screws must be left loose until the legs are firmly placed and then tightened.

The instrument must be sheltered from both wind and sun. If the region affords no material that is readily available for constructing wind screens and sun shelters a folding wind screen and a sun umbrella must be carried as a part of the regular outfit.

Before observations are begun at a station all adjustments of the theodolite must be tested and such as are found in error must be corrected, special attention being paid to the micrometers to eliminate errors of run. The stations to be sighted must next be carefully identified by means of the directions shown on the plat or by means of angles previously taken with a prismatic compass. If any of the distant stations can not be seen with the unaided eye some object in line with each which can be found quickly must be selected, or, if necessary, the direction to each may be marked by some object near by, so no time shall be lost in making the pointings when the angles are being read.

Method of observing.—With micrometer theodolites either single angles may be measured or the method of circle readings (directions) may be adopted. In using the latter method select for the initial point some station that is especially distinct and easily sighted, and use it as the initial point for all sets of readings. The telescope being set on the initial point, read both micrometers, then sight the other stations in succession in the order of their azimuths (clockwise rotations), closing on the initial point. Then reverse telescope, set on initial point, and sight the stations in reverse order. This completes one set of readings with telescope direct and reversed. Now shift the circle about 36° (*examine the plate bubbles after this shift and relevel if necessary*) and commence another set. When pressed for time it is advisable to shift the circle when telescope is reversed. No angle should be considered as well determined that has not been measured on at least 5 different parts of the circle or 10 times in all, 5

with telescope direct and 5 with telescope reversed. When the telescope is reversed each end of its axis will rest in the same Y as before. Reversals are of especial importance when there is an appreciable difference in the elevations of the points sighted.

If the observations are made in the afternoon it is advisable to take all secondary pointings before commencing the observations to stations, and there should be at least two sets of such pointings; the remaining time for observing can then be devoted to the accurate measurement of the important angles while conditions are the most favorable.

The graduated circle should never be placed so that when pointing at any particular station the micrometers will be set to even degrees except, as before noted, while data are being obtained for "reduction to center."

Field record.—The field record is to be kept in book 9–912. It must be written in a plain, neat hand, with a No. 4 pencil, or with ink and no part of it must on any account ever be erased. A single line should be drawn through erroneous records, the corrected figures being written above. If deemed necessary an explanation should be written in the column for remarks. The memory should not be trusted for data of any kind; the record must be faithfully kept in all particulars and be made so complete that it can be understood by another person at any time.

The flyleaf of each notebook must be properly filled in when the book is first used and one of the blank flyleaves must contain an index of the contents.

The date, name of station, time of observing, and names of observer and recorder should be systematically entered at the head of each page.

The position of the instrument with respect to the center of the station must be clearly defined and if it is set up off the center a full statement must be given of the distance and the angles measured.

On the page immediately preceding the record of angles should be written a minute and complete description of the station occupied, the station marks, character of signal, nearest camping or other stopping places, roads, and trails, also a statement regarding the ownership of the land and such other information as will be helpful to

the topographer. The description must be written before the recorder leaves the station and should be accompanied by a rough diagram showing directions to other stations and plan indicating location of instrument if it was not centered on the station.

Inasmuch as station names are to be published, effort should be made to select names that have local significance.

Reading and recording of angles.—When the micrometer wires are set for a reading with the Geological Survey theodolites it is very important that the last movement of the wires be toward the right. The readings on the graduated head are then decreasing and the spring attached to the slide which holds the wires is being compressed. If the cross wires are moved the least bit too far to the right they must not be turned backward to the setting but must be turned backward at least a halfturn of the screw, then brought forward slowly to a correct setting. When the setting is properly made a division on the graduated plate will appear exactly midway between the two movable cross wires and an equal amount of white space will show on each side of it. A part at least of the micrometer adjustment errors can be eliminated by making the settings with less than five turns of the screw; this can always be done if the right-hand part of the comb scale is sometimes used for comb scale and micrometer head readings, the 10-minute space being taken from the left.

For all precision instruments where a tangent screw and spring are used together, the setting should be made while the spring is being compressed, otherwise the "slack" of the screw may cause an error.

The recorder should not only take down the readings called off by the observer but should without delay compute the angles between successive stations and also the mean readings. The following form is to be used for recording angles by the method of directions:

Station occupied, *Ivy*.
Date, *May 3, 1910*.

Observer, *R. D. J.*
Recorder, *C. P. M.*

Stations sighted.	Microm. A.	Microm. B.	Mean.	Angle.	[In these columns write summaries of the angles measured.]		
	° ′ div.	° ′ div.	° ′ ″	° ′ ″			
Telescope direct.							
Bald Knob	*350 48 02*	*170 48 00*	*350 48 02*	....	*Bald Knob–*	*Shumate–*	*Time 4:30 p. m. All signals distinct.*
(Station mark)	*000 00 15*	*180 00 14*	*(000 00 29)*	....	*Paint:*	*Burning*	
Paint	*77 54 10*	*257 54 10*	*77 54 20*	*87 06 18*	*87 06 18*	*Rock:*	
Shumate	*212 40 05*	*32 40 04*	*212 40 09*	*134 45 49*	*21*	*63 25 35*	*Eccentric point occupied, distant from station mark 2.20 meters:*
Burning Rock	*276 05 24*	*96 05 20*	*276 05 44*	*63 25 35*	*23*	*30*	
Walnut	*285 51 19*	*105 51 17*	*285 51 36*	*9 45 52*	*20*	[*etc.*]	
Workman	*320 47 10*	*140 47 09*	*320 47 19*	*34 55 43*	*16*		
Bald Knob	*350 47 29*	*170 47 27*	*350 47 56*	*30 00 37*	*22*		
Telescope reversed.					*21*		*87 06 20.5*
Bald Knob	*170 47 18*	*350 47 20*	*350 47 38*	*30 00 39*	*25*		*134 45 50.4*
Workman	*140 46 28*	*320 47 01*	*320 46 59*	*34 55 41*	*20*		*63 25 32.0*
Walnut	*105 51 08*	*285 51 10*	*285 51 18*	*9 45 50*	*19*		*9 45 51.6*
Burning Rock	*96 05 13*	*276 05 15*	*276 05 28*	*63 25 30*			*34 55 42.0*
Shumate	*32 39 28*	*212 40 00*	*212 39 58*	*134 45 49*	*87 06 20.5*		*30 00 41.*
Paint	*257 54 03*	*77 54 06*	*77 54 09*	*87 06 21*	*Paint–Shu-*		
(Station mark)	*180 00 10*	*000 00 12*	*(000 00 22)*		*mate:*		*359 59 57.5*
Bald Knob	*170 47 22*	*350 47 26*	*350 47 48*		*134 45 49*		
					49		
					51		
					53		
					48		
					46		
					50		
					52		
					54		
					52		
					134 45 50.4		

[NOTE.—Two readings on the station mark only need be taken for reduction to center.]

[Left-hand page.] [Right-hand page.]

Opposite each angle record any necessary information as to visibility of signals or atmospheric conditions.

Field computations.—Angles at each station should be reduced to center in the field in order to test the triangle closures, which for a primary scheme should not exceed five seconds.

Arbitrary adjustments and preliminary computations of positions should also be made in the field. Book 9–889 is to be used for summary of angles and for miscellaneous computations. Computations for distances should be entered in book 9–901 and for coordinates in book 9–902. For field computations of coordinates where the lines are short five or six place logarithms will give sufficient accuracy and the computations may be shortened by omitting some of the minor corrections, carrying results to tenths of seconds of latitude and longitude only.

Triangulation plot.—A careful plot of the work should be kept on the scale of 10 miles to an inch, and each month a reduced copy, on which angles measured are indicated by the usual sign, should be sent in on the monthly-report blank. The plot, if carefully made, will prove invaluable for finding directions to distant stations. Place the protractor on the plot with 0° in line with a station that can be seen clearly, then read in turn the angle to each other station, thus obtaining an observing list.

Azimuth observations.—There must be not less than two azimuth stations in each triangulation scheme, but if the azimuth of any line in a scheme can be computed from former observations then only one azimuth station need be established for each square degree controlled.

The azimuth mark should be placed at least half a mile from the station. It should consist of a vertical slit one-fourth to one-half inch wide and 6 inches long, cut in a small box containing a candle or lantern. To illuminate the cross wires of the instrument and to read the angles, an electric hand lamp is to be preferred.

The observations should consist of not fewer than five direct and five reversed measurements between the star and mark. As the star is at a much higher angle of elevation than the mark it is important that the horizontal axis of the theodolite be adjusted with care and leveled. The ends of the striding level bubble must be read at each setting on the star and a level correction computed if there is an appreciable difference between them, as shown in the example attached.

Observations on Polaris should be made immediately preceding and following elongation, as any error in the time of observation has then the least effect on the resulting azimuth. The time of setting the cross wires on the star must be recorded to the nearest second. The watch error must be known and to this end the triangulator should compare his watch frequently with telegraphic time, which is sent over Western Union lines once a day, usually at noon Washington time.

Example of record of azimuth observations.

Station: Canada, Ky. 8-inch theodolite No. 434. One division of micrometer= 2″. One division of level=2″ of arc. June 11, 1910. Watch 0^{m} 23^{sec} slow.

Telescope direct.

Object.	Time a. m.	Level. West.	Level. East.	Micrometer. A.	Micrometer. B.	Mean.	Angle.
	H. m. s.	*Div.*	*Div.*	° ′ *div.*	° ′ *div.*	° ′ ″	° ′ ″
Mark.......	....	......	......	352 54 09	172 53 21	172 54 00	
Polaris.....	2 30 50	11.0	10.0	288 35 24	108 35 05	108 35 29	
		9.0	12.0				
		20.0	22.0	....	....	....	64 18 31
		−2.0					

Telescope reversed.

Object.	Time a. m.	Level. West.	Level. East.	Micrometer. A.	Micrometer. B.	Mean.	Angle.
Mark.......	....	......	......	172 53 04	352 52 13	352 52 47	
Polaris.....	2 36 48	11.0	10.0	108 34 10	288 33 27	288 34 07	
		9.0	12.0				
		20.0	22.0	....	....	....	64 18 40
		−2.0					
Polaris.....	2 40 26	10.0	11.0	202 41 13	22 40 22	22 41 05	
Mark.......	....	10.0	11.0	138 22 24	318 22 06	318 22 30	
		20.0	22.0	....	....	....	64 18 35
		−2.0					

Telescope direct.

Object.	Time a. m.	Level. West.	Level. East.	Micrometer. A.	Micrometer. B.	Mean.	Angle.
Mark.......	....	......	......	22 41 07	202 40 17	202 40 54	
Polaris.....	2 46 52	10.0	11.0	318 22 20	138 22 13	138 22 33	
		11.0	10.0				
		21.0	21.0	....	....	....	64 18 21
		0.0					

NOTE.—Four other sets should be taken.

COMPUTATIONS.

Preliminary computations of distances from unadjusted angles should be made in the field as required by the rule on page 19.

The steps in the final adjustment and computation are as follows:

1. Computation of mean angles.
2. Closing the horizon.
3. Tabulation of angles.
4. Reduction to center.
5. Tabulation of triangles which form geometric figures.
6. Computation of spherical excess.
7. Formation of side or sine equations.
8. Formation of equations of condition.
9. Formation of table of correlates.
10. Formation of normal equations.
11. Solution of normal equations.
12. Substitution of corrections.
13. Correction of tabulated angles and sines.
14. Distance computation.
15. Computation of geodetic coordinates.
16. Tabulation of results.

Operations 1 and 2 are completed in field record book 9–912; 3 to 5 and 7 to 13, inclusive, in book 9–889; 6 and 14 in book 9–901; and 15 in book 9–902. The results are tabulated on printed blanks 8 by 10½ inches in size, one blank for each station.

Closing the horizon.—In careful work closing errors will always be small and may be distributed among the various angles in proportion to their number. If there are any angles measured which should equal the sums of smaller angles, proper corrections must be made before the horizon is closed.

For convenience of reference a rough plot should be made for each station on part of a page in book 9–889, showing relative size and position of the various angles with names of stations sighted, and on the same or the following page should be given a summary of all the angles at the stations, in order of azimuth, with the angles and distances to signals for eccentric stations.

Reduction to center.—For eccentric stations the data for reduction to center should be indicated on the plat and figures given for them in the summary. An illustration of the method of procuring these data is given below. (See also fig. 1, p. 24.) Two sets of angles were read at Elk station (where an eccentric point was occupied), with one of the micrometers set very nearly on 0°, when the

telescope was pointing directly toward the center of the signal. The angle to each point in turn is given below. By measuring the angle with this setting the computer is saved considerable trouble, and the possibility of error is lessened. The measured distance between the center of the instrument and the center of the station was 4.7 feet (1.43 meters).

The formula for computing the swing in seconds for any line is—

$$\frac{\text{Distance to signal}}{\sin 1''} \times \frac{\text{sin angle signal to far station}}{\text{Distance to far station}}$$

The distance to signal will be a constant for each set up, hence its logarithm may be combined with the sine of 1 second and this constant used throughout the computation. The distances to the distant stations in logarithms of meters are derived from a preliminary computation.

$$\begin{array}{ll} \log 1.43 & = 0.15534 \\ \log \sin 1'' & = 4.68557 \\ \hline \log \text{constant} & = 5.46977 \end{array}$$

ELK STATION.

Station	Dick.	Taylor.	Browning.	Tweedy.
Angle	23° 07′ 10″	68° 43′ 40″	109° 16′ 54″	206° 27′ 10″
Log constant	5.46977	5.46977	5.46977	5.46977
Log sin angle	9.59400	9.96935	9.97493	9.64881
A. C. log distance	5.70154	5.59196	5.74781	5.63275
Log correction	0.76531	1.03108	1.19251	0.75133
Correction in seconds	+5.83	+10.74	+15.57	−56.41

The sign for any correction is the same as that for the sine of the angle, therefore for an angle over 180° it will be negative.

The correction for any angle will be the difference between the corrections for the two lines bounding it, always taking the lines in order of azimuth. Thus, for Dick–Elk–Taylor it will be—

$$\begin{array}{r} +10.74 \\ -\ 5.83 \\ \hline +\ 4.91'' \end{array}$$

For Browning–Elk–Tweedy it will be—

$$\begin{array}{r} -56.41 \\ -15.57 \\ \hline -71.98'' \end{array}$$

The general rule is, change the sign of first correction (in order of azimuth) and add algebraically to the second correction. The sum will be the correction to the angle. The angles listed on page 25 have all been corrected.

The foregoing formula may be used also when it is desired to compute the "swing" for a line, which is to be applied at a distant station to change the pointing to the marked point—that is, the station center—from that taken to the signal. Whether the computed swing is to be added to or subtracted from a given angle may easily be found by an inspection of the diagram.

Formation of triangles.—By an inspection of the field plat of the triangulation determine what groups of triangles are so interrelated that a change in one will affect the others and what groups of triangles should be adjusted as a unit. For the triangulation by the Geological Survey, which is not executed for geodetic purposes, it is not advisable ever to include more than 15 or 20 triangles in such a group, because the labor of solving equations for the adjustment of any group increases rapidly with its size.

Four overlapping triangles form the simplest group that may be adjusted by the usual least-square methods.

Assume the group shown in figure 1 for adjustment. Tabulate the angles for each triangle, as shown at (a), (b), (c), and (d) (p. 25). Any angle in any of these triangles may be considered as the difference between the azimuths (directions) of its two sides. For example, angle Dick–Elk–Taylor, or 3.0.2, using for convenience the figures assigned to each angle vertex, would be the azimuth or direction of the line 3–0 subtracted from the azimuth or direction of the line 2–0. Azimuths are always measured in a clockwise direction. Therefore this angle may be indicated as $-3.0+2.0$ or $-3/0+2/0$. In the latter form the denominator is always the figure at the vertex of the angle and with the vertex pointing toward the observer the left-hand direction is always given the minus sign. (Directions will hereafter be referred to as sides.)

Spherical excess.—For any triangle on the earth's surface the sum of the three angles, if correctly measured, will exceed 180° by an amount varying with the area. For considerable areas the observed angles must be reduced to their plane values by deducting one-third

the spherical excess from each. The spherical excess for any triangle between latitude 25° and 45° is approximately 1 second for each 75.5 square miles of area, or exactly equals in seconds ABm sin C, in which A, B, and C are respectively the lengths of the two sides in meters and the included angle of any triangle, and m is a constant depending on the latitude. The logarithms of m are given on page 271, "Geographic tables and formulas." In computing spherical excesses for any figure (as that on p. 25 and fig. 1, for example) arrange the work systematically, the logarithms of each of two sides in meters from a preliminary computation, the logarithm of the

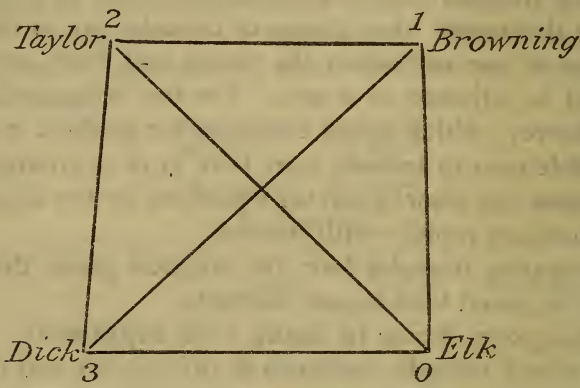


FIGURE 1.

sine of their included angle, and the logarithm of m for the mean latitude for each triangle; place in a column. Give the figures for the triangle at the head of the column, as 3.0.2, using the angle 3.0.2 and the sides 3–0 and 2–0 in the computation.

Triangles.	3.0.2	2.0.1
Log side A	4.29846	4.40804
Log side B	4.40804	4.25219
Log sin C	9.85406	9.81304
Log m	1.40475	1.40475
Log spherical excess	9.96531	9.87802
Spherical excess in seconds	0.92	0.76

Mean latitude, 37° 35′.

In the same manner the spherical excess for each of the remaining triangles is computed.

As the spherical excess for a given area is constant, the sum of the spherical excesses for the triangles 2.0.1 and 3.0.2 must equal the spherical excesses for the other two. This check should always be applied to the results. In many cases it will be convenient to perform this computation in the book (9–901) used for preliminary distances, in the left-hand column adjacent to each triangle.

Angle equations.

Stations.	Side.	Observed angle.	Correction.	Corrected spherical angle.
		° ′ ″	″	° ′ ″
(a) Elk	−2/0+1/0	40 33 19.17	+2.12	40 33 21.29
(a) Browning	−0/1+2/1	95 23 07.62	+ .71	95 23 08.33
(a) Taylor	−1/2+0/2	44 03 30.52	+ .62	44 03 31.14
		179 59 57.31		180 00 00.76
		.76		Spherical excess.. 0.76
		Error −3.45		
(b) Elk	−3/0+2/0	45 36 34.90	−2.97	45 36 31.93
(b) Taylor	−0/2+3/2	50 34 37.57	− .40	50 34 37.17
(b) Dick	−2/3+0/3	83 48 53.15	−1.33	83 48 51.82
		180 00 05.62		180 00 00.92
		.92		Spherical excess.. 0.92
		Error +4.70		
(c) Elk	−3/0+1/0	86 09 54.07	− .84	86 09 53.23
(c) Browning	−0/1+3/1	50 10 30.58	−1.47	50 10 29.11
(c) Dick	−1/3+0/3	43 39 38.99	− .43	43 39 38.56
		180 00 03.64		180 00 00.90
		.90		Spherical excess.. 0.90
		Error +2.74		
(d) Dick	−2/3+1/3	40 09 14.16	− .90	40 09 13.26
(d) Browning	−3/1+2/1	45 12 37.04	+2.18	45 12 39.22
(d) Taylor	−1/2+3/2	94 38 08.09	+ .21	94 38 08.30
		179 59 59.29		180 00 00.78
		.78		Spherical excess.. 0.78
		Error −1.49		

Sine equation.

		Sides.	Angle.	Sine.	Difference for 1″.	Correction in seconds.	Correction to sine.	Corrected sine.
(e)	+	−2/3+0/3	83 48 53.15	9.9974645	+02.2	−1.33	− 3	9.9974642
		−2/0+1/0	40 33 19.17	9.8130350	+24.6	+2.12	+52	9.8130402
		−3/1+2/1	45 12 37.04	9.8510731	+20.9	+2.18	+46	9.8510777
				9.6615726				9.6615821
	−	−2/3+1/3	40 09 14.16	9.8094543	+24.9	− .91	−23	9.8094520
		−3/0+2/0	45 36 34.90	9.8540576	+20.6	−2.97	−61	9.8540515
		−0/1+2/1	95 23 07.62	9.9980787	− 2.0	+ .71	− 1	9.9980786
				9.6615906				9.6615821
				Error −180				

Equations of condition.

(f) $0=-3.45''-2/0+1/0-0/1+2/1-1/2+0/2$

(g) $0=+4.70''-3/0+2/0-0/2+3/2-2/3+0/3$

(h) $0=+2.74''-3/0+1/0-0/1+3/1-1/3+0/3$

(i) $0=-1.80''-.022\ 2/3+.022\ 0/3-.246\ 2/0+.246\ 1/0-.209\ 3/1+.209\ 2/1+.249\ 2/3$
$\qquad -.249\ 1/3+.206\ 3/0-.206\ 2/0-.020\ 0/1+.020\ 2/1$

$0=-1.80''+.227\ 2/3+.022\ 0/3-.452\ 2/0+.246\ 1/0-.209\ 3/1+.229\ 2/1$
$\qquad -.249\ 1/3+.206\ 3/0-.020\ 0/1$

Table of correlates.

(j)	(k)	(l)	(m)	(n)	Correlates after substituting computed values.					
Sides.	1	2	3	4	1 +0.275	2 −0.063	3 −0.589	4 +3.007	Corrections.	Sides.
1/0	+1	.....	+1	+0.246	+0.275	....	−0.589	+0.740	+0.426	1/0
2/0	−1	+1	....	− .452	− .275	−0.063	....	−1.359	−1.697	2/0
3/0	.....	−1	−1	+ .206	....	+ .063	+ .589	+0.619	+1.271	3/0
0/1	−1	.....	−1	− .020	− .275	....	+ .589	−0.060	+0.254	0/1
2/1	+1	.....	....	+ .229	+ .275	....	....	+0.689	+0.964	2/1
3/1	.....	.....	+1	− .209	.......,	....	− .589	−0.628	−1.217	3/1
0/2	+1	−1	....	....	+ .275	+ .063	....	....	+0.338	0/2
1/2	−1	.....	....	....	− .275	....	....	....	−0.275	1/2
3/2	.....	+1	....	....	....	− .063	....	....	−0.063	3/2
0/3	.....	+1	+1	+ .022	....	− .063	− .589	+0.066	−0.586	0/3
1/3	.....	.....	−1	− .249	....	....	+ .589	−0.749	−0.160	1/3
2/3	.....	−1	....	+ .227	....	+ .063	....	+0.683	+0.746	2/3

(o) *Normal equations.*

1	2	3	4	Absolute term.
+6.000	−2.000	+2.000	+0.947	−3.450
....	+6.000	+2.000	−0.863	+4.700
....	....	+6.000	+0.122	+2.740
....	....	....	+0.51779	−1.800

Solution of equations.

(p) Normal equations.

	1	2	3	4	Absolute term.
(p1)	+6.000	−2.000	+2.000	+0.947	−3.450
(p2)	(.1667)	+ .333	− .333	− .1578	+ .575
(p3)		+5.333	+2.667	− .5473	+3.550
(p4)		(.1875)	− .500	+ .1026	− .666
(p5)			4.000	+ .0800	+2.115
(p6)			(.250)	− .0200	− .529
(p7)				+ .31060	− .934
				(3.2196)	+3.007

(q)

	2	3	4	Absolute term.
(q1)	+6.000	+2.000	−0.863	+4.700
(q2)	− .667	+ .667	− .3157	−1.150
(q3)		+6.000	+ .1220	+2.740
(q4)		− .667	− .3157	+1.150
(q5)		−1.333	+ .2737	−1.775
(q6)			+ .51779	−1.800
(q7)			− .14944	+ .544
(q8)			− .05615	+ .364
(q9)			− .00160	− .042

(r)

	1	2	3	4
(r1)	+0.575	−0.666	−0.529	+3.007
(r2)	− .475	+ .309	− .060	
(r3)	.196	+ .294	− .589	
(r4)	− .021	− .063		
(r5)	+ .275			

Least-square adjustment.—After deducting the spherical excesses from the sums of angles for each triangle (a), (b), (c), (d), page 25, the differences between the remainders and 180° will be the errors, plus for remainders over 180° and minus for those less than 180°.

The rules for determining the number of angle equations and the number of sine or side equations required for the proper adjustment of any figure are these:

$$L - S + 1 = \text{angle equations}$$
$$L - 2S + 3 = \text{sine equations}$$

Where L equals number of lines in the figure and S the number of stations. A solution of these equations for a quadrilateral shows that three angle equations and one side equation are required. In the present example it is immaterial which three triangles are used for the adjustment.

To select the sines for the side equation: Consider the figure as a pyramid with vertex at 2; by redrawing the figure with the line 3–1 dotted and the triangle 2–3–0 shaded, it will appear to the eye as such a pyramid. Select for the first set of angles for the sine equations those opening to the front in going around the base of the pyramid from 3 to 0 to 1 to 3; mark them with solid arcs of circles; the remaining angles around the base make up the other set and are marked with dotted arcs. In selecting the point for the vertex of the pyramid, as a general rule choose the one which includes the smallest angles, but if all the angles are greater than 30° either station may be chosen. Find the sines for each set of angles, recording also the differences for 1″ for each; call the first set of sines plus and the second set minus, find the difference between them, and give it the sign of the greater.

Equations of conditions are now made up as follows: For triangle (a), error equals −3.45″; this is made up of the errors in the azimuth or pointing of the sides $-2/0+1/0-0/1+2/1-1/2+0/2$, six in all. In like manner form equations (f), (g), and (h). The sine equation (i) is made up as follows: The error of the sines, being the difference between the two sets, is −180. To correct the sines, changes in seconds to be found for the angles must be multiplied by the differences for 1″ in column 4 of (e) for the given angle; hence for the first sine this will be +2.2 multiplied by the corrections to be given the directions −2/3 and +0/3, or if expressed in a simple form it will be $-2.2\ 2/3+2.2\ 0/3$. Treat each side and difference for 1″ in like

manner, noting, however, that for the second set of sines, which is considered negative, each sign given for the side will be reversed; for example, the first one is written +24.9 2/3−24.9 1/3. It will be noticed that in the first form of (i) as written, 2/1 appears twice with like signs, 2/3 appears twice with unlike signs; combine like terms algebraically, thus reducing the equations to the second form of (i). For the convenience of the computer and in order to avoid the handling of large numbers, both equations (i) have been divided through by 100; this, of course, does not alter their value.

There are now four equations to be solved and twelve unknown quantities; the latter are combined and reduced to four in the table of correlates. Column (j) contains the marks for the sides or directions for which corrections are required. Column (k) contains on the proper lines the algebraic coefficients for the various sides from equation (f); for example, −2/0, considered as a quantity, might be written −1 (2/0), and +1/0 in like manner written +1 (1/0) −1 and +1 are therefore the entries for column (k), lines 2/0 and 1/0.

The formation of normal equations from the table of correlates is as follows: Column 1 of (o) contains the sum of the squares of each quantity in column (k). Column 2 contains, first, the product of each quantity in column (k) by corresponding quantities in column (l); second, the sum of the squares of each quantity in column (1). Column 3 contains the sum of the products of (k) by (m), (l) by (m), and (m) by (m) (the squares). Column 4 is made up in same manner, using the quantities and signs as given. If columns 1, 2, and 3 were completely filled out by products found as indicated above, it would be found that the quantities from +6.000 down the column would be the same as those from +6.000 along the lines to the right to column 4. But as the former are not needed in the solution they have been omitted; had they been retained the equations in full would be as follows, the second member of each equation being zero:

Normal equations.

1	2	3	4	Absolute term.
+6.000	−2.000	+2.000	+0.947	−3.450
−2.000	+6.000	+2.000	− .863	+4.700
+2.000	+2.000	+6.000	+ .122	+2.740
+ .947	− .863	+ .122	+ .51779	−1.800

These are ordinary algebraic equations which may be solved by the usual rules of algebra, but as the solution of 5, 10, 15, or more equations is often required in Geological Survey work the process should be conducted systematically as shown.

The first normal equation is written in full on line (p_1), parts of the other equations are written on lines (q_1), (q_3), and (q_6). The reciprocal from Barlow's tables of the first quantity (+6.000, line (p_1), column 1), is placed at the left. The product of this reciprocal (0.1667) by the quantities on line (p_1), columns 2, 3, and 4, and absolute are written immediately under each in turn; the quantity +0.333 (line (p_2), column 2) is now used as a multiplier for line (p_1) (omitting column 1), and the products are placed in columns 2, 3, 4, and absolute, line (q_2); in like manner the quantities −0.333 (column 3, line p_1) and −0.1578 (column 4) are used as multipliers and the products written on lines (q_4) and (q_7). The algebraic sums of lines (q_1) and (q_2) are now written on line (p_3), which is then used as if it were an original equation. The reciprocal of +5.333 is found and used as a multiplier as before and the products written on line (p_4). The next products are written on lines (q_5) and (q_8). The sum of each column of lines (q_3), (q_4), and (q_5) is carried over to (p_5). The process is repeated for each equation until finally the product +3.007 is found, which is the value for unknown quantity numbered 4. This value and also the quantities in the column of absolute terms, lines (p_6), (p_4), and (p_2), are copied in table (r), line (r_1). With +3.007 as a multiplier products of each quantity in column 4, lines (p_6), (p_4), and (p_2), are found and written on line (r_2), columns 3, 2, and 1. Column 3 of (r) is then summed and the result (−0.589) is the value of unknown quantity numbered 3. This is used as a multiplier and products found with quantities from columns 3 and 2, lines (p_4) and (p_2), and in like manner values for unknown quantities numbered 2 and 1 are found.

The solution of the normal equations and the values found for the unknown quantities may be checked, if desired, by substituting in the full equations, page 29, but usually an experienced computer will not need to undertake this extra work, depending rather on the accuracy of his results until checked by use as correction in the original triangles, (a), (b), (c), and (d), and in the sine equation (e).

The next step in the adjustment is to substitute the values for the four unknown quantities in the tables of correlates and to find the correction for each side. The method of doing this can be easily seen by following the process through the right-hand half of that table. For convenience, the value found for each unknown quantity is written at the head of columns 1, 2, 3, and 4. Each of these in turn is multiplied by quantities in columns 1, 2, 3, and 4 of the left-hand part of the table and the products are placed in the right-hand part on the same line with the multiplicand. The final correction for any side is then the algebraic sum of the quantities which are on line with the side number in columns 1, 2, 3, and 4 (at the right side of the table). Thus the correction 1/0 is made up of $+0.275$, -0.589, $+0.740 = +0.426$; this is the correction in seconds to the side. The correction for any angle, then, is the difference between the corrections for the two sides bounding it. For example: Angle at Elk, triangle (a), is:

$$\begin{array}{rr} -\text{ correction } 2/0 = & +1.697 \\ +\text{ correction } 1/0 = & +\ .426 \\ \hline & +2.123 \end{array}$$

The correction for any sine is the correction for the corresponding angle multiplied by the difference for $1''$ in the sine.

It is desirable to have triangles close without errors greater than a hundredth of a second and sine equations close to the seventh place of logarithm, but unless the normal equations are carried to three or four decimal places, there will possibly be residual errors of two or three hundredths in some triangle closures. It is, however, considered allowable to make arbitrary changes of not over $\pm 0.03''$ in angles in order to procure consistent results.

Large figures.—Generally adjustments must be made of figures larger than quadrilaterals. The equations on page 28 show how many angle and sine equations are required for any figure. By inspection, large figures may be divided into simple groups, each line of which must be a necessary part of a pyramid, as suggested on page 28. The number of pyramids will be the same as the number of sine equations, and no more. Select from each pyramid group the triangles which would be used in adjusting that group alone, remembering that triangles so selected must always cover the given area once, but never the entire area twice. After making up a series of sine and angle equations in this manner, if any triangle appears

in several groups, omit each duplication of it, checking the final number retained by the formula. It is sometimes advisable to omit a long, unimportant cross line in a large figure rather than increase the labor of adjustment by retaining it. When loss of accuracy will not result, groups of triangles should be adjusted as units which will not require more than 8 or 10 normal equations. For United States Geological Survey work groups involving over 15 equations should not be undertaken.

Weighted adjustments.—In order to adjust as a whole an extensive triangulation scheme, the strongest groups are adjusted first; then if lines or triangles in them form parts of other groups, their first adjusted values are given infinite weights and thus left unchanged. Weights if used in an adjustment appear in the table of correlates only. For any side they are used by inserting them in an extra column in the table (this would be placed between (j) and (k) if used in the present example). The weight for a side taken as a whole number 1, 2, 3, etc., or ∞, is written on the line with the side number. Any product with quantities on that line in either the right or the left side of the table is divided by the corresponding weight before it is used in any way; where ∞ is used this results in canceling all of the corresponding products—in other words, the side may be omitted.

Computations of distances.—These are to be made in book 9–901. The triangles are arranged in order from a given base or known side, one page or part of a page being taken for each new station. For each triangle the adjusted spherical angles and the spherical excess are given to hundredths of seconds.

Station.	Spherical angle.	Spherical excess.	Plane angle.	Log sines and distances.
			9.8853615	4.4789254
	° ′ ″		° ′ ″	
Browning	50 10 29.11	0.30	50 10 28.81	0.1146385
Elk	86 09 53.23	.30	86 09 52.93	9.9990263
Dick	43 39 38.56	.30	43 39 38.26	9.8390917
	180 00 00.90		180 00 00.00	
Miles, 16.827, Browning to Elk				4.4326556
Miles, 24.319, Browning to Dick				4.5925902

The rule for the solution of plane triangles for which the three angles and one side are given is that the sides are proportional to the sines of the opposite angles. By always arranging the angles in the above form with the new station first, the solution is made somewhat mechanical. The logarithms of the sines of plane angles are, of course, used; that for the angle at the new station from which distances are required to the other two stations is written immediately above the angle; its arithmetical complement (10 minus the sine) is written to the right and on line with the angle. Each of the other sines is placed on line with the angle to which it relates. Immediately above the sines is written the logarithm of the distance in meters between the second and third stations in the triangle; in the example this is 4.4789254 for the line Elk to Dick.

To get the logarithm of the distance from the new station (Browning in the example) to one of the other stations, omit the sine opposite the latter and add together the remaining logarithms in the right-hand column. The distance to thousandths of a mile for each computed line must be found and placed to the left of the names of the terminal stations. The work should be verified by comparing distances for each line that has been computed from two or more triangles.

Computation of geodetic coordinates.—For this work use book 9–902 and check results by computing each position from the two stations which form a triangle with the new station. For convenience, only one of the computations is here given:

	° ′ ″
Azimuth a: *Elk–Dick*	*96 56 01.12*
Spherical angle at *Elk*	*86 09 53.23+*
Azimuth a′: *Elk–Browning*	*183 05 54.35*
Δa+180°	*180 00 36.47*
Azimuth (a): *Browning–Elk*	*3 06 30.82*

Geodetic Coordinates.

LATITUDE. ° ′ ″			LONGITUDE. ° ′ ″
ϕ	*37 28 47.32*	*Elk.*	λ *82 00 16.16*
$\Delta\,\phi$	*14 37.10(+)*		$\Delta\,\lambda$ *0 59.77(−)*
ϕ'	*37 43 24.42*	*Browning.*	λ' *81 59 16.39*

Geodetic Coordinates—Continued.

Computation for latitude:

log s........	*4.4326556*
log B.......	*8.5110415*
log cos a'...	*9.9993647(−)*
log (I)......	*2.9430618(−)*
log s^2.......	*8.86531*
log C.......	*1.28943*
log $\sin^2$ a'...	*7.46561*
log (II).....	*7.62035*
log D.......	*2.3771*
log $[I+II]^2$..	*5.8861*
log (III)....	*8.2632*
log E.......	*6.0528*
log $s^2 \sin^2$ a'.	*6.3309*
log (I)......	*2.9431(−)*
−log (IV)...	*5.3268(+)*

△ *arc and sine*	
(*log s*)	−13
(*log* V)	+00
	−13

φ 37	*−28−*	*47.3*
φ' 37	*43*	*24.4*
72		*11.7*
37	*36*	*06*

Computation for longitude:

log s..........	*4.4326556*
log sin a'.....	*8.7328074(−)*
log A'........	*8.5091756*
log sec φ'.....	*0.1018382*
△ a and s....	(*−13*)
log (V).......	*1.7764755(−)*
	59.769−
△λ...........	*59.77−*

Computation for azimuth:

log (V')........	*1.776476(−)*
log sin $\left(\frac{\phi+\phi'}{2}\right)$	*9.785450*
log sec $\left(\frac{\Delta \phi}{2}\right)$	*0.000000*
log (VI)........	*1.561926(−)*
−△a...........	*36".47(+)*

Computation for latitude:

(I)..........	*877.126(−)*
(II).........	.*004* +
(III)........	.*018* +
(IV)........	.*000(+)*
	877.104(−)
−△φ.......	*14' 37".10(+)*

[I+II]	*877.122*
log [I+II]	*2.94306*
log $[I+II]^2$	*5.88612*

Azimuth check.

	°	'	"
	53	*16*	*59.94*
	3	*06*	*30.82*
Check:	*50*	*10*	*29.12*
Spherical angle at *Browning*	*50*	*10*	*29.11*

Computation of Azimuth a, in Book *2752*, page *21*.
Spherical angle and distance=s, in Book *2751*, page *9*.

Computed by *E. M. D.*

[NOTE.—Logarithms marked + or − are found from the signs of the cosine and sine of azimuth.]

In this example the azimuth at the top of the page is derived from a previous computation. The spherical angle is that at Elk from the adjusted figure. Whether to add or subtract this can be

determined very easily by inspecting a plat of the stations, but when for one of the pair of computations the spherical angle is added, the other is always subtracted. The latitude and longitude at Elk are also derived from a previous computation. Logarithm s is the logarithm of the distance in meters between Elk and Browning. The constants B, C, D, and E are from "Geographic tables and formulas" for the known latitude at Elk. Cosine a′ and sine a′ are functions of the azimuth Elk to Browning. The algebraic sign of each of these as fixed by trigonometric rules determines the sign of the resulting quantity. The signs of (II) and (III) are always positive; that for (IV) is always opposite to that of (I). The constant A′ and secant ϕ' in the longitude computation are for the new latitude, which requires that the latitude computation be made first. These two factors will be the same for each of the pair of computations for the new position. For short lines, corrections (III) and (IV) will usually be less than 0.01″ and may be neglected.

When the logarithm of distance *s* in meters exceeds 4.0000000, a correction will usually be required for logarithm (V) for the difference between the arc and sine. The constants for computing this are given on page 269, "Geographic tables and formulas," the arguments being log distance *s* and log (V). The difference between the values found from these is always to be subtracted from log (V) before finding its value in seconds.

Six places of decimals will usually give sufficient accuracy for log (VI). The logarithm of secant $\left(\frac{\Delta\phi}{2}\right)$ may be taken from page 268 of the tables. When log (V) is large, say over 3.5000000, a correction in seconds will be needed for Δa expressed by the factor $\Delta\lambda^3$ F. The logarithm of (V) is multiplied by 3 and added to the logarithm of F, which is given in the tables; the value in seconds for the resulting logarithm is always to be added to the previously found value in seconds for (VI).

The latitudes and longitudes for each point thus computed in pairs should agree within one or two hundredths of a second. The difference between the two reverse azimuths should also agree with the corresponding adjusted spherical angle within one or two hundredths of a second.

The final step in the computation of triangulation is the tabulation of the results. A printed blank is used; on it is written the name of the station, the State and county in which it is situated, the kind of signal and the center mark used, a full description of the station (see p. 16), the latitude and longitude, the azimuth, back azimuth, and logarithm of distance in meters to all other stations from which it is visible, also for each logarithm of distance the corresponding distance in miles and thousandths.

Reduction of azimuth observations.

Canada, Kentucky, triangulation station. June 11, 2.30 a. m., 1910 (civil date).
Latitude: 37° 35′ 46″. Longitude: 82° 21′ 39″.

	H.	*m.*	*s.*
Watch time of observation 2^h 30^m 50^s a. m. = 14^h 30^m 50^s of the astronomic day which commenced at noon June 10	14	30	50
Correction from seventy-fifth meridian time to 82° 21′ 39″ correction for 7° 21′ 39″ (p. 111, "Geographic tables and formulas")		−29	27
Watch slow by telegraphic time		+	23
Local mean time (astronomic day)	14	01	46
Correction, mean to sidereal time (p. 113, "Geographic tables and formulas," or p. 591, "Nautical almanac")		+ 2	18
Right ascension of mean sun at Greenwich noon, June 10, corrected for 5^h 29^m 27^s to change to noon at 82° 21′ 39″ west longitude ("Nautical almanac," June, Table II; change for longitude made by Table III, last part of almanac)	5	12	45
Sidereal time of observation	19	16	49
Right ascension of Polaris for nearest Washington transit—June 10.8 ("Nautical almanac" circumpolar stars for June)	1	26	12
Hour angle of Polaris at time of observation	17	50	37
Hour angle of Polaris in arc = t (p. 112, "Geographic tables and formulas")	267°	39′	15′,

The daily change in Polaris is so slight that for the purpose of this computation no account need be taken of a fraction of a day in computing its position.

The following are the formulas for azimuth and level correction:

$$\tan A = -\frac{a \sin t}{1 - b \cos t}. \quad a = \sec \phi \cot \delta. \quad b = \tan \phi \cot \delta$$

$$\text{Level correction} = -\frac{d}{4}[(w+w') - (e+e')] \tan h;$$

in which—

ϕ=latitude of station (37° 35′ 46″).
A=azimuth of Polaris at time of observation.
δ=declination of Polaris at time of observation (88° 49′ 21″).
t=hour angle of Polaris at time of observation (267° 39′ 15″) (both the sine and cosine of this angle are negative for this example).
d=value of one division of level (2.0″).
w, w'=readings of west end of level bubble, direct and reversed.
e, e'=readings of east end of level bubble, direct and reversed.
h=angular elevation of star (at elongation this is equal to the latitude, nearly).

The following is the computation of the first of the preceding observations (page 20):

Level correction$=\frac{2}{4}(2\times 0.77)=0.77''$

Log tan ϕ		9. 88649
Log cot δ		8. 31290
Log cos t		8. 61205 (negative).
Log b cos t		6. 81144 (negative).
b cos t	−	1. 000648
		1
1−b cos t		1. 000648
Log sec ϕ		0. 10109
Log cot δ		8. 31290
Log sin t		9. 99964 (negative).
Log a sin t		8. 41363 (negative).
Log 1−b cos t		0. 00028
Log tan A	−	8. 41335
A		1° 29′ 01.″7
Level correction		+ 0. 8
		1 29 02. 5
Add 180° to refer to the south	=	180
Angle star to mark		64 18 32
Azimuth of mark		245° 47′ 34″. 5

Each azimuth computation should be made in a single column and for convenience the columns should be placed side by side in tabular form.

GENERAL SUGGESTIONS TO COMPUTERS.

Do not crowd your work; paper is cheap.

Do your work in a systematic manner. If it permits tabular arrangement always use the forms approved by other computers

unless you can convince them that yours are better. The Survey has printed forms for many purposes; these should be used whenever possible, for by their use the work is made more mechanical, and the more mechanically the work is done the less chance there is of error.

A computer who is inexperienced or out of practice should check his work in every way possible. He should check logarithms either of numbers or of circular functions by using first a tabular value for a quantity less than the given one and then a greater tabular value, so that the differences in one case may be added and in the other subtracted. This operation may be reversed when the logarithm is given and numbers or angles are required.

Errors are frequently made in taking out the first three figures of a logarithm from the wrong line where a dash over the fourth figure indicates that the first three should come from a lower line.

As the algebraic signs of cosines and sines are so frequently required, the rules governing them should be firmly fixed in the mind; as an aid to this remember the general rule that distances measured upward or to the right on the conventional plat of the quadrants of the circle are considered positive, others negative. The wrong use of signs is a very common source of error.

Each step in a long computation, if it is not at once automatically checked, should be checked by repeating the computation.

Check the copying of angles, distances, etc., taken from adjusted results for use in new computations; also check figures carried from page to page.

Gross errors are sometimes made by using the sine when a cosine is required, or by writing a product in the wrong column, as east for west in primary traverse computations.

Placing the decimal point in the wrong place is a common mistake. This may in many cases be corrected by a mere inspection of the quantity to see whether it appears of proper value.

Good judgment should be exercised in the degree of accuracy sought for a given result. For the preliminary computation of geodetic positions, for example, six-place logarithms will probably suffice; these can be taken from a seven-place table with only a rough interpolation. A four-place logarithm can often be used to advantage. The accuracy of the results obtained should equal the requirements; more than this involves a waste of time.

For convenience the foot, yard, and mile are the units adopted for all Geological Survey field work, but for geodetic computations meters are used. The best conversion tables for metric and English measures are those published by the Bureau of Standards, edition of 1910. In using these all changes from one system to another should be checked by reversing the operation. The logarithms for the interchange of these measures are given on page 301 of "Geographic tables and formulas."

PRIMARY TRAVERSE.

FIELD WORK.

Personnel of party.—In primary traverse the party consists of an instrument man in charge, a recorder, two tape men, and two flagmen; also a cook and a teamster when camping is necessary.

Instruments, notebooks, etc.—The following supplies can be obtained on requisition:

One transit, graduated to 20 or 30 seconds, and furnished with stadia wires.
Two 300-foot steel tapes, graduated to feet throughout.
One 100-foot steel tape.
Two red and white transit rods.
Two plumb bobs.
Eleven tally pins.
Four hand recorders.
Two electric hand lamps.
One tape repair outfit, punch, and rivets.
Three tape clips, temporary repairs.
Two tape holders.
One set steel dies, figures.
One set steel dies, letters.
Three large book bags.
Standard bench-mark tablets or posts (according to the requirements of the country).
Canteens.
Cement (in cans).
Drills, hatchet, hammer, post-hole digger.
Primary traverse field notebooks 9-928.
Tape men's notebooks 9-929.
Blank notebooks 9-896.
Book of instructions.

The instrument man must carry a reliable watch.

Location of line.—Primary traverses should always be run in circuits or tied to points previously located. In 15-minute quadrangles, in country where routes can be readily planned, traverse lines should follow as closely as possible the borders of the quadrangles to be controlled, not departing from them more than is absolutely necessary to keep on roads. If there is a choice of roads select the one in unmapped areas. An additional line should be run to bisect the quadrangle.

In areas where the country will not permit this plan to be followed economically and where the selection of routes for the lines must be influenced by the location of highways, it will be necessary to plan the routes to meet the specific requirements.

Permanent marks.—In regions where topographic conditions permit, tablets or iron posts (see *C* and *F*, Pl. I) must be set as near as possible to each corner of each 15-minute quadrangle, one on each side halfway between the corners and one in the center, making nine in all. All such marks must be stations on the line and should be stamped "Prim. Trav. Sta. No. —" (and numbered consecutively) and also with the year of survey. In areas which can not be traversed according to the regular plan, permanent marks must be established at intervals not greater than 6 miles.

In cooperating States use the appropriate State post or tablet (*A*, Pl. I).

Where level bench marks have been established along the route of survey, they should be tied to and stamped as above and thus made to serve as permanent marks on the traverse line.

It is desirable that every permanent point be tied to two or more witness or reference points, the bearings, distances, and descriptions being duly recorded in the notebook.

Secondary points.—Besides the permanently marked points, a number of other points should be carefully located along the traverse, and these points should be specifically designated in the field notes. Of special importance are the crossings of boundaries of States, counties, and civil townships, and the locations of the principal cross roads, of railroad stations when the line follows a railroad, and of township and section corners if the region is subdivided by public-land surveys. Note should also be made of less important landmarks,

such as road forks, mileposts, railroad switches, road and stream crossings. These points should be so completely described in the notebook as to be readily identified.

Duties of tape men.—The front tape man carefully marks off each tape length; if on a wagon road, with tally pins; if on a railroad, with keel on the rail. Each time he marks off a tape length he registers it on his hand recorder; each time the rear tape man reaches the mark left by the front tape man he does likewise. When a transit station is established the two tape men compare their hand recorders for check on tape length. Should they differ, the course must be remeasured.

Transit stations should be made at even tape lengths or even 10-foot marks, wherever possible, in order to simplify the work of the computer. They should be selected at points affording not only an unobstructed view back to the transit but also a clear view forward. Each station is to be marked, if on a wagon road, by a 10-penny nail driven into the ground and through a piece of paper on which the front tape man has written the number of station and distances; if on a railroad, by a keel cross on rail, with number and distance on nearest tie.

Stations on main lines are to be numbered consecutively, beginning with zero; those on short spur lines to section corners or other points to be computed are to be lettered instead of numbered. Station numbers should never be duplicated in a single locality.

The two tape men must keep in book 9–929 separate records of the number of stations and distances between them. At noon and at night these records must be compared with the recorder's notes, and should there be a difference, it must be corrected before the line is carried forward, the line being retraversed if necessary.

In locating transit stations the front tape man should bear in mind that it is desirable for the instrument man to be able to sight the bottom of the rod in each direction. This is especially important on short sights, for errors due to sighting the upper part of a rod which may be out of plumb may appreciably affect the accuracy of the line.

Method of measuring.—When measuring along a wagon road the tape must be kept horizontal unless the grade is very slight; on

steep slopes a plumb bob must be used either to bring the tape end vertically over an established point or to establish a new one, as the case may be. Judgment should be used in selecting the proper length of tape on slopes. No attempt should be made to use the full 300-foot length; about 150 feet is ordinarily all that a tape man can hold horizontal with the proper tension and plumb at the same time. On slopes that require "breaking" the tape into short sections, the entire tape should first be drawn forward its full length by the front rodman if convenient, or by the front tape man, who then returns to help "break" the tape at the proper places, until the end of the tape is reached. In this manner the distance is measured on the whole tape and does not depend on the sum of the separate horizontal measurements.

Some tension must be put on the tape, but the use of a spring balance has been found by experience to be unnecessary.

Errors in taping.—The errors that most seriously affect the accuracy of taped lines may be classed under two heads.

The errors of one class are due to failure to keep the tape horizontal and to careless plumbing. The instrument man should impress tape men with the fact that the accuracy of traverses depends on their correct taping more than on the instrument work, for the latter is checked at every azimuth observation, whereas there is no check on the taping until the circuit is closed.

The errors of the second class are gross mistakes arising generally from carelessness in counting tape lengths. They may be eliminated by checking the count of tape lengths by independent measurements. To do this, the instrument man should read each distance by stadia on the red and white transit rod or on a special stadia rod carried for this purpose. In case the distance is too great to be read by a single sight, he should set up the transit between stations and read both front and rear rods. Stations should in no case be more than 2,600 feet apart, which is about the limit of visibility of the rod. On railroads an additional check on the taping may be had by counting rail lengths. This should be done by both rodmen and by the recorder, or by the instrument man while moving from one station to the next. In other places a check may be had by pacing.

Method of reading deflection angles.—At each station the instrument man should proceed as follows: Sight rear flag with transit circle set at last reading at previous station, transit telescope, sight front flag, and read both verniers. Turn instrument with the two plates clamped, the vernier remaining undisturbed; sight rear flag again and remeasure the angle. If the two results thus obtained differ more than 60″, repeat the operation.

When the transit is carried from one station to the next, keep the upper plates clamped so as to retain the last vernier reading; after setting up the instrument verify the reading and use it as the first back sight reading at the new station. It may at times be necessary, in order to get the best pointing on the rod, to change the reading a minute or two, but by following this general plan a useful check on the readings is secured without trouble.

Azimuth observations.—Observations on Polaris for azimuth must be made at the close of each day's work, if the weather permits. On a crooked line with many short courses azimuth stations should be not more than 100 stations apart; on a traverse with long tangents they should fall not more than 15 miles apart. These requirements may necessitate going back over the line in order to make the necessary observations, but if conditions are favorable it is possible to make azimuth observations in broad daylight.

Both the transit and the azimuth mark must be at stations in the traverse not less than 500 nor more than 1,500 feet apart. Each point should be marked by a stake with a tack, or, if, on a railroad, by a nail in a tie. The azimuth mark may consist of a vertical slit one-eighth inch wide and 6 inches long cut in the side of a box or tin can containing a candle or lantern, which should be carefully centered over the tack in the stake. In pointing the telescope use the electric hand lamp to illuminate the cross wires, holding it nearly in front of the object glass.

Angles should be read as follows: Set on azimuth mark, then on star; reverse telescope, set on star again, and then on azimuth mark. Each observation should consist of not less than three direct and three reversed measurements, the circle being shifted for each set by about 60°. (See sample page of record, p. 44.) Observations may

be made at any time the star is visible, but preferably when at or near elongation. The time of setting the cross wires on the star must be recorded to the nearest second. Observations should be made rapidly; not more than 10 minutes need be taken to complete a set. The notes must be kept in the following form:

Date, Sept. 10, 1910. Line from Pikeville west to Dayton, Mo.

Azimuth observation 2.5 miles southeast of Dayton, Mo., Sept. 10, 1910. Mag. dec. sta. 326–327 N. 57° 30′ W. Lat. 39° 00′. Long. 92° 15′.

Instrument at station 326. Mark at station 327. Watch 35 seconds fast, ninetieth meridian time.

Point.	Vernier A.	Vernier B.	Mean.	Deflection angle.	Azimuth.	Time.
	° ′ ″	° ′ ″	° ′ ″	° ′ ″	° ′ ″	*H. m. s.*
Mark..........	216 54 00	36 54 00	36 54 00			
Star[a]..........	274 48 00	94 48 00	94 48 00	57 54 00	....	8 31 33
Star...........	94 50 30	274 50 30	94 50 30	57 54 30	....	8 34 48
Mark..........	36 56 00	216 56 00	36 56 00			
Mark..........	348 02 30	168 02 00	168 02 15			
Star[a]..........	45 57 00	225 56 30	225 56 45	57 54 30	....	8 40 28
Star...........	225 57 00	45 57 00	225 57 00	57 54 45	....	8 41 50
Mark..........	168 02 00	348 02 30	168 02 15			
Mark..........	95 05 00	275 06 00	95 05 30			
Star[a]..........	153 01 30	333 01 00	153 01 15	57 55 45	....	8 43 55
Star...........	333 02 30	153 03 00	153 02 45	57 56 30	....	8 44 56
Mark..........	275 06 30	95 06 00	95 06 15			
				57 55 00		8 39 35
				Watch fast........		35
				Corrected time.....		8 39 00

[a] Reverse telescope between each two readings on star.

The latitude and longitude of each azimuth station, scaled from the best map available to the nearest minute, should be given, together with the date of observation, on the page with the other records, in order to enable the computer readily to convert standard to local mean time.

In case unfavorable weather prevents the taking of the azimuth, leave adequate marks at a point selected, before proceeding with the line, and return to them later to make the observations.

Watch error.—The instrument man must carry a reliable watch and keep it in good condition. He should ascertain its error daily by comparison with telegraphic time, which is sent over Western Union lines once a day. In case he has no opportunity to make this comparison while running the line, he should do so as often as possible, figure the rate of error per day, and record the proper correction for each azimuth observation made. A watch error of 20 seconds or less will not appreciably affect the accuracy of the determination. At least once in each notebook he should state whether he uses standard time; if so, for what meridian.

Magnetic declination.—A careful reading of the needle for magnetic declination should be made at frequent intervals and recorded opposite the proper station number in the notebook. Such determinations should be made at each azimuth station and at favorable points along the line where the needle is not likely to be affected by rails, electric wires, or similar disturbing elements. At azimuth stations determine the magnetic bearing of the azimuth mark at the time it is established. If the line follows a railroad, magnetic determinations should be obtained from a parallel line at a distance of 25 yards from the rails or wires.

Field record.—Complete notes must be kept by the recorder in book 9–928, to be written in a plain, neat hand with a No. 4 pencil. The blanks in the title-page should be filled in the first day the book is used. A single line should be drawn through erroneous records, which must never be erased.

The recorder must take down the vernier readings, as they are called off by the transit man, and compute the mean pointings and deflection angles, giving proper signs to the latter. He must keep up with the instrument man in these computations, as they enable him to note by inspection whether the instrument man has made errors in his readings and to call attention to them before the instrument is removed from the station. He should take special pains to see that the degree and minute numbers for the two verniers are consistent and are recorded in the proper column.

The notes are to be kept in the following form:

Date, Sept. 9, 1910. Line from Pikeville to Dayton, Mo.

Stations, distance between.	Vernier A.	Vernier B.	Mean.	Deflection angle.	Azimuth.	Remarks.
	° ′ ″	° ′ ″	° ′ ″	° ′ ″	° ′ ″	
Sta. 326: 3 tapes, 900 feet.	316 51 30	136 52 30	316 52 00		[a]123 35 00	Sta. 326–327; N. 57° 30′ W.
	275 06 00	95 07 30	275 06 45	41 45 15		
	233 21 00	53 22 00	233 21 30	41 45 15	....	Stadia 905.
				−41 45 15	[b]81 49 45	
					[c]81 49 47	
Sta. 327: 4 tapes + 120 = 1,320 feet.	233 21 30	53 22 00	233 21 45	45 43 15		
	279 04 30	99 05 30	279 05 00	45 44 00	....	Stadia 1,330.
	324 48 30	144 49 30	324 49 00			
				+45 43 37	[b]127 33 22	
					[c]127 33 26	
Sta. 327+90 feet, stream crossing.						
Sta. 327+430 feet, crossroad at Tanbark P. O.						
Sta. 328: 260 feet.	324 48 30	144 49 30	324 49 00	17 19 30		
	342 08 00	162 09 00	342 08 30	17 19 00		
	359 27 00	179 28 00	359 27 30		....	Stadia 250.
				+17 19 15	[b]144 52 37	
					[c]144 52 43	

[a] Written in red ink. [b] Written with black pencil. [c] Written with black ink.

NOTE.—The entries in the azimuth column are a part of the office computation.

The record must contain also a description of the starting and ending points of the line, of each permanent mark established along the line, of each point which is to be computed for the use of the topographer, and of all crossings and other landmarks that may be of value to him. Such descriptions should be concise, yet full enough to leave no possible doubt as to the identity of the points described. Each should be supplemented by an explanatory sketch if necessary.

Example of description of permanent mark:

Station 1025, bench-mark tablet stamped "Prim. Trav. Sta. No. 4, 1910," set in sandstone ledge, top of Walden Ridge, 3 miles northwest of Dayton, Tenn., at junction of Dayton, Pikeville, and Morgan Springs Roads, 325 feet west of residence of John Neilson. Reference marks: Cross cut in ledge 60.25 feet N. 25° 30′ E.; spike in root of white oak tree 14 inches in diameter, 75.60 feet N. 45° 15′ W.

Examples of description of points to be computed and other landmarks:

Station 625+730 feet [center of crossroads at Antioch Church].
Station 720+320 feet, east abutment of bridge over Glade Creek.
Station 732+—— feet, road fork at Johnson blacksmith shop.
Station 926+210 feet [center of track opposite semaphore, Lee station].
Station 936+300 feet, road crossing one-half mile east of Sequatchie railroad bridge.

Each point to be computed should be marked with brackets in ink immediately upon its selection by the instrument man.

COMPUTATIONS.

The steps in primary traverse computations are as follows:

1. Computation of azimuths.
2. Computation of azimuths of each line from the observed deflection angle.
3. Adjustment of closing errors of azimuth.
4. Computation and tabulation of latitudes and departures, which are the north and south distances and the east and west distances, by two computers working independently.
5. Latitude and longitude computation.
6. Adjustment of closures in positions.
7. Tabulation of results by atlas sheets.

The computations for 1, 2, and 3 are made in the original field record book, 9-928; those for 4, 5, and 6 are in book 9-931. The abstracts of results (7) are placed on long sheets of blank paper.

Reduction of observations on Polaris for azimuth.—First find the mean of time of observations and corresponding mean of angles measured between mark and star (p. 44). Having ascertained the approximate latitude and longitude of the azimuth station, compute the true azimuth of star by tables given on pages 14 to 25, inclusive, "Geographic tables and formulas."[1] For any hour angle or latitude not given in Table 3, pages 20 and 21, the azimuth to 0.1 minute can usually be mentally interpolated.

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[1] Commencing with the year 1911 the General Land Office will issue yearly tables of azimuths of Polaris giving declination, elongation, and culminations for each day in the year. These tables should be used whenever available in preference to those in "Geographic tables and formulas." Be sure to note whether tabular culminations are for a. m. or p. m.

Example of computation: The station is in latitude 39° 00′ N., longitude 92° 15′ W.

	H.	m.	s.
Sept. 10, 1910, ninetieth meridian standard time of observation	8	39	00
Correction for 2° 15′ longitude		− 9	00
Local mean time of observation	8	30	00
Add 24 hours, thus finding the interval from noon Sept. 9, because Table 1 for September gives upper culmination at an hour greater than 8^h 30^m.	24	00	00
	32	30	00
Upper culmination (Table 1), Sept. 1	14	41.9	
Correction to reduce to year 1910 (Table 1A)		+ 3.0	
Correction (Table 1B) for 9 days		−31.5	
Upper culmination, Sept. 9, 1910	14	13.4	
Time of observation	32	30	
Hour angle	18	16.6	
This hour angle, being greater than 11^h 58^m, indicates that the star was east of north. In order to enter Table 3 this hour angle must be subtracted from 23^h 56^m.1	23	56.1	
Time argument for Table 3	5	39.5	

For which the azimuth found by a double interpolation is 90′, east of north.

To which 180° is to be added to refer to south.

To interpolate for hour angles near elongation, use for the latter 5^h 55^m and take the corresponding angle from the line next below the five-hour time given (or the line next above if more than six hours), which is the yearly mean elongation angle; or, more accurately, take the elongation angle from Table 2 corrected for the month. When using Table 3, 0.5 should be added wherever a period is given after the figures for minutes.

	°	′	″
Azimuth of star (90′ east of north)	181	30	00
Angle between mark and star (p. 44) (star east of mark)	57	55	00
Azimuth at station 326 to 327	123	35	00

A rough check on this azimuth is found by comparison with the observed magnetic bearing, allowance being made for declination. Each azimuth computation is to be made in the field notebook on the same page with the observation.

This azimuth is written in red ink in the azimuth column of field notebook (see example on p. 46) on the line with the station number.

The deflection angle is added or subtracted according to its sign, and the sum or remainder is written in pencil on the line with the mean deflection angle. The next deflection angle is combined according to its sign with this azimuth and the result placed in pencil opposite the deflection angle used. This process is repeated until the next computed azimuth written in red ink is reached. The last azimuth in pencil will probably not agree with the observed azimuth. For any line not running due north or south there will be a discrepancy between observed and computed azimuths, due solely to convergence of meridians, which for latitude 30° will be 0.5′ for each mile run east or west; for latitude 49° the amount will be 1′. If no large errors appear in the results, the discrepancy between computed and observed azimuths at the second station is to be divided by the number of stations and a proportional correction applied to each penciled azimuth, the corrected figures being written in black ink.

Latitudes and departures are to be computed in book 9–931 as shown below:

Line from Pikeville to Dayton, Mo.

Station.	Azimuth.	Distance.	Sine.	Cosine.	North.	South.	East.	West.
	° ′ ″							
326—327........	81 49 47	900	0.990	0.142	....	128	....	891
327+430........	127 33 26	430	.793	.609	262	....	....	341
					262 128	128	....	1,232
					134			
327+430—328...	127 33 26	890	.793	.609	542	....	....	706

Natural sines and cosines for the azimuth given are written in the proper columns. By means of Crelle's tables the products of these by distances are found and placed in the proper columns. The sines multiplied by the distance give departures east or west; when the sine is positive the new point is west, when negative it is east. Cosines multiplied by distances give latitudes north or south; when the cosine is negative the new point is north, when positive it is south. The direction of the new point can readily be determined by

noting the azimuth, remembering that 0° azimuth is for a line running due south, 90° for a line due west, 180° for a line north, and 270° for a line east; in the example 81° 49′ 47″, being for an azimuth between due south and due west, will be to a point southwest. For long distances four decimal places in sines and cosines should be used. Whenever a point is reached for which the latitude and longitude are desired, as at 327 + 430 in the example, leave six blank spaces for the computation. The data for the computation for such a point are found from the record on page 46, as follows: For the crossroad at Tanbark post office, which is on line between stations 327 and 328, the azimuth is the same as to station 328. The distance by measurement is that given, 430 feet from station 327. In order to make the computations continuous, station 328 is taken as 1,320−430=890 feet from the intermediate point used, the azimuth being the same for both points.

The next step in this work is the computation of latitudes and longitudes. These should be determined for important points a mile or less apart. Assume, for illustration, that for station 326 (p. 46) the coordinates have been computed, and that 327+430 is the next location desired; each of the four columns, north, south, east, and west, is summed and the difference between the sums of the north and south columns is placed in the column of the greater; likewise, the difference between the east and west columns is placed in the column of the greater. The computations of latitude and longitude and the descriptions of the points are placed on the right-hand page of the book opposite the group of stations.

The logarithms of the geodetic constants for metric measures, called "the A, B, C factors," are on pages 196 to 267, inclusive, of "Geographic tables and formulas." Factors A and B are used to five decimal places only; these will be practically constant for a distance of 10 or 15 miles north and south, the value for the middle latitude being used.

For the example on page 49:

Log distance 134 (north)	2.12710
Log to reduce feet to meters	9.48402
Log B for latitude 39° 00′ 00″	8.51093
	.12205

The sum, 0.12205, is the logarithm of change in latitude in seconds between station 326, and 326+430=1.32″ (north).

For change in longitude:

Log distance 1,232 (west)	3.09061
Log to reduce feet to meters	9.48402
Log A for latitude 39° 00′	8.50914
Log secant of middle latitude	.10950
Log of change in longitude in seconds	1.19327
New point west	15.61″

These differences are to be added to the latitude and longitude of station 326.

In order to check the plotting the distance between successive positions must be computed, as the lines are seldom as much as a mile in length and never over 2 miles, the latitude and departure can with sufficient accuracy be taken as the base and perpendicular of a plane triangle. The distance sought will then be the hypothenuse and its square will be equal to the sum of the squares of the base and altitude. For distances less than 10,000 feet Barlow's tables should be used in finding squares or square roots. The distance should be written in red ink, inclosed in a circle, on the right-hand page of the computation book in the blank space between the two stations referred to. After the record is complete its accuracy should be tested by computing a side from the given distance (hypothenuse) and the other side.

These operations are repeated for each selected point until the traverse line closes back on itself or ties to another point previously determined. The errors of closure for a 15′ quadrangle, if not in excess of 1″ in latitude or 1¼″ in longitude, may be distributed proportionately between initial and closing points.

Where so many operations are involved errors are very likely to creep into the computations. Therefore each step of the work should be checked as well as possible. The azimuth computation should be compared with the observed magnetic bearings, but because of the possibility of local variation little dependence can be placed on this comparison as a check. If the computed and observed azimuths for a line differ about 10′, look for an error of that amount in the deflection angle or in the adding and subtracting

of deflection angles to azimuths. If the difference is larger it is very likely that a wrong sign has been used for a deflection angle. To find the error, divide the difference by 2 and look for a deflection angle with an incorrect sign equal to the quotient. Errors of about 180° occasionally result from the recorder placing the vernier readings in the wrong columns. By a careful inspection of the records it is sometimes possible to detect such an error. The latitudes and departures should be computed by two persons working independently of each other; after each has completed his work the results should be compared and differences corrected and verified. Errors are often made in multiplication by the distance, the decimal point being in the wrong place, or the product is written in the wrong columns—in the north column when it should be in the south column, etc.

VERTICAL CONTROL.

PRIMARY AND PRECISE LEVELING.

GENERAL INSTRUCTIONS.

Distribution of primary-level control.—A sufficient amount of accurate spirit leveling should be done to insure the placing of at least two standard bench marks in each township or equivalent area surveyed, except in forest-clad or mountain areas, where at least one such mark should be placed in each township.

Permanent bench marks should be established along level lines at intervals of approximately 3 miles, unless otherwise instructed, and in no case should the distance between bench marks exceed 6 miles.

Location of permanent bench marks.—Bench marks should be established, if practicable, at the township corners of the public-land surveys, near all important lakes and reservoirs, at the crossings of important streams and divides, in every city or town passed through, and in the vicinity of important mines. They should be so located as not to be liable to injury or disturbance, yet should be so prominently situated that they can easily be found. Along a railroad or highway bench-mark posts if used should be placed either outside of and close to the right of way or on the right-of-way line. They must not be set close to trees, telegraph poles, or fence posts.

Character, setting, and marking of permanent bench marks.—Standard bench marks consist either of tablets fastened with cement into solid rock in place or into masonry structures, such as the foundations of buildings or bridge piers, or of iron posts set in the ground so as to project not more than 1 foot and surrounded by a conical mound of earth about 3 feet in diameter raised to half the height of the post.

Portland cement in air-tight cans is furnished from the Washington office for use in setting tablets. If good clean sand is available it can be mixed with the dry cement in equal parts. The drill hole for the tablet must be well cleaned and wet. The cement and sand, or cement alone if pure sand can not be conveniently procured, should then be thoroughly mixed with water to a thick paste, and the drill hole filled with it; into this the tablet should be pressed, the excess cement being forced out so as to completely fill the space under the tablet face. In order that the cement may set well it should be kept damp and protected from the sun for at least a day, and it must not be allowed to freeze for 12 hours. Dry earth or a piece of sacking will probably be sufficient protection. When a tablet is set in a vertical wall it may be necessary to hold it in place by a prop of some kind for a few hours.

The intersection of the cross lines on either style of mark is the reference point. Before a tablet is set the figures indicating the elevation (to the nearest foot only) are to be stamped into the metal before the word "feet."

In cooperating States the name of the State must be stamped or cast on standard bench marks.

If a tablet is inconspicuously situated, a mound of rock should be erected near it, the rock about it be marked with paint, or a near-by tree blazed as a witness tree.

The steel tape can often be used to advantage instead of the leveling rod for determining the elevation of a tablet set in a vertical wall.

All standard bench marks should be used as turning points in the line, but where this can not be done, *their elevation must be determined by two readings from different set-ups, or from separate temporary bench marks.*

Temporary bench marks.—Temporary bench marks should be set at intervals ranging from half a mile to 1¼ miles. They may consist of chiseled marks on solid rock or masonry, or copper nails with washers, or spikes, driven in telegraph poles, mileposts, fence posts, or trees. The copper nails with lettered washers must be used when practicable. Where there are no natural objects for temporary bench marks, pieces of iron pipe, about 20 inches long, may be used. Select a place where the mark will not be likely to be disturbed and yet can be readily found, preferably near a road junction, so it will afford a convenient tie point for other levels or traverses. The location should be conspicuously indicated by large figures in white or red paint, thus:

U S
[elevation]
B M

Useful elevations.—Besides the bench marks above described a number of intermediate elevations are required for the use of the topographer, and these also should be selected with a special view to their usefulness in topographic mapping. The levelman should bear in mind that his work is not an end in itself but a preparation for the work of others, and that the accuracy with which his circuits check, though of paramount importance, is not the only thing that determines its utility.

Ground elevation should be painted conspicuously along the side of the road, on fences, telephone poles, trees, or rocks. If practicable, they should all be marked on the same side of the road, preferably on the north or east side.

The points at which elevations are particularly desired are the top of the rail at railroad stations, junctions, sidings, and crossings, the ground at crossroads, road forks, and bends; on summits and ridges; near schoolhouses and other public buildings, lone houses, and important mines, quarries, and oil, gas, and artesian wells; on some permanent part of a bridge other than a wooden floor; the water surface of streams under bridges, at stream crossings, and above and below dams; and water surfaces on lakes and reservoirs. Where water-surface elevations are recorded, always give the date.

The number of these elevations should be varied with the nature of the country and the contour interval; thus in rugged regions

mapped with 50-foot or 100-foot intervals relatively few elevations are required (mostly on summits and in hollows); but in areas of gently rolling relief they should be more numerous. In flat areas where 5 or 10 foot contours are used, each contour crossing should be marked with a stake or otherwise. This is important, as in such areas a difference in elevation of a few tenths of a foot may mean a difference of several hundred feet in the location of a contour.

Descriptions of bench marks and useful elevations.—Complete descriptions of all bench marks and useful elevations must be made in the notebook and copied in the description book (9–916) at the close of each day's work. A sketch must accompany the description of each standard bench mark, showing directions and distances to near-by objects.

Descriptions should be written with items in the following order:

1. Name of the nearest post office, town, village, or other well-known locality, with direction and distance from it to the bench mark in miles and tenths; or township, range, and section in which bench mark stands, with direction and distance from nearest corner.

2. Position with reference to buildings, bridges, mileposts, street or road corners.

(Items 2 and 3 should be written in direct form of speech.)

3. Description of object on which the bench mark is placed—tree, bowlder, bridge, etc.

(The above three items answer the question *where* and should be followed by a semicolon (;) and by item 4, which answers the question *what*.)

4. Nature of the bench mark—copper nail with washer, bolt, mark on rock, tablet, post, etc.—and how marked or stamped. Old bench marks must be fully described.

Descriptions should be kept in the order in which the bench marks occur. If standard bench marks are not established when the line is first run, spaces should be reserved in description books for them in their proper order. A brief description of the line should be given at frequent intervals, especially when changing direction. When circuits are closed, complete descriptions of closing points, closure error, old and new elevations, and page reference to connecting points should be given. A plot of all lines or circuits must be made

on a page near the back of the description book for each group of circuits and the names of enough places to identify the line readily should be added. Boundaries of quadrangles should be shown, and also, if the area is covered by public-land surveys, the position of the line with reference to township and section lines. Alongside of each line reference to the page of the description book where the record is made should be entered. The records in this book are incomplete without this diagram.

PRIMARY LEVELING WITH Y LEVEL.

Personnel.—A primary level party consists of a levelman, one or two rodmen, and in some cases a bubble tender.

Instruments, notebooks, etc.—The instruments required are as follows:

- One 20-inch Y level.
- One or two New York rods.
- One or two plumbing levels.
- Two steel turning pins.
- One set dies (figures and letters).
- One 25-foot steel tape.
- Bench-mark tablets or posts.
- Copper nails and washers for temporary bench marks.
- Cement in cans.
- Level notebooks 9–903 (those in black covers to be used by levelmen; those in yellow covers by rodmen).
- Bench-mark description book 9–916.
- Two book bags.

Other accessories to be purchased in the field:

- One or two hatchets.
- One drill hammer.
- One posthole digger.
- Stone drills (1⅛-inch bit).

Character of lines.—Primary levels should be run as single lines in circuits wherever practicable, otherwise checked by rerunning, preferably in the opposite direction. No work is completed until it is checked in some way. Lines should be connected with near-by bench marks of railroads, cities, and other organizations.

Accuracy.—The allowable closure error of a circuit in feet must not exceed

$$0.05\ \sqrt{\text{length of circuit in miles.}}$$

If it is greater than this, the facts must be reported to the geographer in charge immediately.

Adjustment of instruments.—The adjustment of the level must be tested daily and corrected whenever it is found in error; the adjustments of the line of collimation and of the level tube are especially important.

The tripod clamping screws must be loosened before the instrument is set and tightened after the legs are firmly placed. After setting the target and before the "all right" signal is given the level bubble should be examined, and if found to be away from center it must be corrected and the target reset.

Equalization of fore and back sights.—In order to eliminate instrumental errors and errors caused by curvature and refraction, it is very important that the length of fore and back sights be equalized, but when this is impracticable as soon as the obstacle is passed enough unequal sights to balance should be taken, provided this can be done before a readjustment of the level is made. When the adjustment of the level is changed, further attempts to eliminate instrumental errors by the balancing of previous sights are useless. The failure to balance sights is one of the principal sources of error.

Maximum length of sight.—The maximum length of sight permissible under the most favorable conditions is 300 feet, except when crossing rivers or deep ravines. In such places proceed thus: Establish a turning point on each side; set up the level about 20 feet from each point in turn, taking in the first position a back sight to the near point and a fore sight to the distant point; then cross the stream or valley and take a back sight to the distant point and a fore sight to the near point. For very long sights several readings should be made on the distant rod and the mean adopted. The mean of these determinations of elevation may be accepted as the true one.

Measuring of distances.—Distances may be measured by stadia readings on the rod, by counting rails if along a railroad, or by pacing. The distances in miles of both fore and back sights must be recorded in notebooks in the proper columns.

Unfavorable conditions.—Work on primary lines should not be carried on during high winds or when the air is boiling badly. During very hot weather an effort should be made to go to work early and remain out late, rather than to work during midday.

Inspection of rod.—When the rod is lengthened beyond 6.5 feet, both the rodman and the levelman must examine the setting of the target as well as the reading of the rod vernier. When the rod is closed they should see that the rod vernier indicates 6.5 feet, not depending on the abutting ends to bring it back to place. The lower end of the rod and the top of the turning point must be kept free from mud and dirt.

Plumbing levels must be tested at intervals and kept in adjustment.

Turning points.—The regular steel turning-point pin should be used wherever no rock or other suitable points are available. A marked point on the top of the rail may be used when running along railroads.

Reading of target.—Both the levelman and the rodman must read each target setting independently and keep separate records. They should not compare figures until their respective records for a given sight are completed. If the difference exceeds 0.001 foot, each must read the rod again before comparing anything but results.

Records.—All level notes must be recorded directly in book 9–903. Under no circumstances should separate pieces of paper be used for figuring or for temporary records. Use ink or No. 4 pencil, make all figures distinct, and do not crowd them. When two important bench marks come close together provide ample room for placing their written descriptions opposite the appropriate figures by dropping the figures for the record one or more lines down the page. For a given H. I. (height of instrument) the rodman's notes must be at least two lines lower down the page than the levelman's and they must not turn over a leaf at the same time. Erasures with rubber or knife are not permissible under any circumstances; a single line should be drawn through an erroneous record and the corrected figures written above it. The flyleaf of each notebook must be properly filled in when the book is first used.

Both the levelman's and the rodman's books must be balanced daily. At the bottom of each page, and at the end of the day's work,

each column of fore and back sight distances and readings should be shown to agree with the difference of elevation previously computed. This check must never be omitted and the computation must appear on the page opposite the notes. Side sights which are not a part of the continuous line should be recorded in an extra column or within brackets.

PRIMARY LEVELING WITH YARD ROD AND PRISM LEVEL.

Personnel.—A prism level party consists of one levelman, two rodmen, a recorder, and an umbrella man.

Instruments, notebooks, etc.—The instruments and outfit consists of the following:

One prism level.
Two yard rods, each to have plumbing level and thermometer attached.
One steel tape (25 feet).
Two steel turning-point pins, hollow head.
One Locke level.
One umbrella with staff.
One set dies (figures and letters).
Bench-mark tablets or posts.
Copper nails and washers for temporary bench marks.
Cement, paint can, keel, and other accessories.
Two book bags.
Prism level notebook 9–940.
Bench-mark description book 9–916.

Character of lines.—Primary levels executed with a prism level need be run in only one direction, but must be in circuits or otherwise checked.

Accuracy.—Circuits must close with an error in feet not exceeding

$$0.04\sqrt{\text{length of circuit in miles,}}$$

which is equivalent to

$$0.056\sqrt{\text{distance between bench marks in miles}}$$

for forward and backward lines.

Graduation of rod.—The rod used is graduated to yards, tenths, and hundredths, and is read by estimation to thousandths. Each yard has a different and distinctive color, which must be recorded for each reading. One edge of the rod has also graduations in feet and tenths for use as a check on yard readings.

Ratio of wire intervals.—The rod is read with each of the three horizontal wires in the instrument. The mean of the two wire intervals in thousandths of a yard as read upon the rod should equal the distance to the rod in feet, but this should be tested. As the upper and lower wires are not always equidistant from the middle wire, the ratio of the wire intervals must be determined from the first day's level notes for use as specified in the next paragraph.

Methods of reading.—The program at each set-up is as follows: After the tripod is firmly set and the clamp screws tightened, level approximately by the circular level, which has been adjusted by comparison with the long level. Point the instrument toward the rod and clamp; bring the level bubble to the center of the tube by means of the micrometer screw. Read on the rod, and first call off the color initials for the lesser and greater extreme readings; second, call yards and tenths for each wire, taking the smallest reading first; third, repeat and read yards, tenths, hundredths, and estimated thousandths; fourth, for additional check on the yard number, read the middle wire on the tenths of feet scale on the back of the rod. Before the level is moved the recorder should first see that the color agrees with the yard readings; second, he must compute the two wire intervals and if their ratio one to the other differs more than 1 per cent from the true ratio (see preceding paragraph), the levelman must repeat the readings; third, he must compute the mean reading in feet by summation, and test units and tenths by mentally multiplying the middle reading by 3, also by comparing with the reading on the scale on the back of the rod. An agreement must be reached before the next sight is taken. The temperature must be recorded for each hour.

Level adjustment.—When the work is commenced, and at least once each day thereafter, the adjustment of the level must be tested by the "peg method" as follows:

At some convenient set-up, after the usual back-sight and fore-sight readings have been recorded, copy the fore-sight on a separate line as a new fore sight apart from the leveling record, leave the fore-sight pin in place, and set a second turning pin about 30 feet back of the instrument; read rod on it for a new back sight; find

from these the mean readings in feet as usual. Move the level forward to a set-up about 30 feet back of the fore-sight pin and take readings on the fore-sight pin and then on the back-sight pin. The constant "C," which is a factor of the adjustment correction, must then be determined thus: Sum of readings on near rods minus that on far rods, corrected for curvature and refraction in feet, divided by three times the difference between the sum of the greater and that of the lesser rod intervals in yards.

The rod interval for any sight is the difference of extreme wire readings.

Example of computation of C.

[To be made in the field.]

Determination of C, 8.20 a. m. August 28, 1910.

Thread reading.	Thread interval.	Sum of thread reading.	Height of instrument.	Sum of thread reading.	Thread interval.	Thread reading.
1.515	0.013				0.105	0.357
1.528	.014				.104	.462
1.542	.027	4.585		1.386	.209	.566
2.252	.105				.012	1.276
2.357	.105				.013	1.288
2.462	.210	7.071		3.865	.025	1.301
	.209	1.385		4.585		
	.419	8.456		8.450	.025	
	.052	—0.0005		8.4555	.027	
	.367 yds.	8.4555	1.101)	—.0055(—0.005	.052	
	3					
	1.101 ft.					

The fraction $\frac{.419}{.002}$=210 feet=sum for far-rod distances.

For correction to be applied to the sum of readings on distant rods for curvature and refraction, see table in back of field book 9–940.

When the sum of the readings on the near rods is the greater, the sign of C will be plus, and vice versa. Great care must be taken in pointing off decimals and in giving proper signs.

Adjustment of bubble.—If the resulting value for C numerically exceeds 0.005, an adjustment should be made by changing the position of the level bubble only, as follows:

Point to a distant rod with the bubble in the middle of the tube and read; move the telescope (by micrometer screws) so as to raise the middle cross wire by an amount which in yards is equal to C times the extreme wire interval. While holding the telescope in this position, bring the bubble to the middle of the tube by raising (or lowering) one end of the level vial with the adjustment wrench; if C is negative, the middle wire must of course be lowered on the rod. After the adjustment has been made, its accuracy should be tested by redetermining the value of C.

In case the cross wires break and the level-tube adjustment has not been disturbed, insert new spider threads and determine a value of C, as above directed. Compare with the last determination of C, and adjust for the difference by changing the position of the ring only—not the level bubble.

Care of instrument.—When the level is on the tripod, be sure that the central tripod clamp screw is tight. Keep the telescope off the micrometer-screw bearing while carrying it between stations. Leave the three tripod wing nuts loose when carrying; clamp tight when tripod is in place for work.

The level must be shaded by an umbrella when in use and by a cloth hood when carried between stations. In rough country the place to set up the rod or level can be quickly found by means of a hand level.

Care of rods.—The rods must always be kept covered when not in use. Never let painted sides touch the ground. Should difficulty be found in holding a rod steady because of wind, two pieces of bamboo or other light poles, 8 feet long, may be held by the rodman against the rod, so as to make a triangular brace against the wind. Plumbing levels must frequently be tested and kept in adjustment.

Testing of rods.—At the beginning and end of the season and at least twice each month during the progress of the leveling the intervals between the metallic plugs on the face of each level rod must be measured carefully in feet to the nearest thousandth, always with the same steel tape, kept for that purpose. The temperature must also be recorded and the number of the tape.

Length of sights.—The length of fore and back sights must be equalized with the prism level as with the Y level. The maximum length of sight with the prism level is 360 feet except at river crossings. Sights across broad river crossings should be taken in the following manner:

Mount the instrument and place stakes so that the center wire will fall near the middle of each rod; if the distance is too great to read the three wires, use improvised targets of cardboard held in place by rubber bands or other simple device, and make several settings by raising and lowering them an equal number of times. Rodmen should be provided with field glasses if necessary to read signals. From bench marks on each bank the elevation of the adjacent water surface should be determined as an additional check.

Record.—The notes are to be kept in ink in book 9–940, as for precise leveling (p. 65), except that each H. I. and level should be computed. No erasures are permitted, either with rubber or knife; a single line should be drawn through erroneous records. Extra fore sights when made should be recorded in the special column on the right-hand page, opposite the H. I., and recorded with "backward," "forward," "right," or "left" added to show the direction to the rod from the instrument.

Computations required must be made at the bottom of each page each night, or oftener if convenient, by both levelman and recorder independently.

On primary work the algebraic sum of the page excesses of back sights or fore sights for each day should be written in the lower right-hand corner of the right-hand page. Three times the sum of the second of each group of three readings in column 1 of the notebook, plus the algebraic sum of the excesses of the lower over the upper thread intervals in column 2, should equal the sum of the mean feet readings in column 3, similarly with columns 7, 6, and 5.

The difference (column 3 minus column 5) should be written at the bottom of column 4 and should equal the difference obtained by subtracting the first from the last elevation, which should be written in the upper space at the bottom of column 4.

The formula 3 C (column 2 — column 6), etc., at the bottom of the right-hand page is for computing the correction to the elevations for combined errors of level and collimation. This computation need not be made in the field. C is the constant which results from the "peg method" test of adjustment. By "(column 2 — column 6)" is meant the difference of the continuous sums of the rod intervals of columns 2 and 6.

PRECISE LEVELING.

For precise leveling the instrumental outfit and the number of men in the party are the same as for primary leveling with prism level, but the following modifications of methods must be made:

Lines must be run independently in both the forward and the backward direction. The allowable error in feet is

$$0.017 \sqrt{\text{distance between bench marks in miles}},$$

and when this limit is exceeded on any section the forward or backward measure is to be repeated until a pair run in opposite directions is obtained between which the divergence falls within the limit. It is especially desirable to make the backward measurement in an afternoon if the forward measurement was made in the forenoon, and vice versa. The observer should make the two measurements under atmospheric conditions as different as possible without materially delaying the work for that purpose. At alternate stations the fore sight is to be taken before the back sight—that is, always take readings on the same rodman first.

The maximum allowable difference between a back sight and the corresponding fore sight mean thread interval is 0.033 yard (33 feet distance). The continuous sums of rod intervals for the section between bench marks must not be allowed to differ more than 0.132 yard (66 feet distance), and they should be kept as nearly equal as possible.

The last set-up of one running must not be copied nor used as the first set-up of a return running—that is, the instrument must be moved so that an independent reading can be obtained.

If any measure over a section differs more than 0.02 foot from the mean, that measure must be rejected. No rejection shall be made on account of a residual smaller than 0.02 foot.

Whenever a blunder, such as a misreading of 1 yard or one-tenth or an interchange of sights, is discovered and the necessary correction is applied, such measure may be retained, provided there are at least two other measures over the same section which are not subject to any uncertainty.

When commencing work for the day and at the beginning and ending of each section record the time. Record the temperature for each set-up, using thermometer readings alternately for each rod,

It is not necessary to complete the H. I. and elevation column, but the difference of elevation for each section should be computed.

The field computations of precise work must be made as the work progresses, on forms provided for this purpose. When original records are completed in the field send them immediately to the chief geographer, Washington, D. C., by registered mail, retaining the corresponding forms until notice is received of the receipt of the original records.

SPECIAL INSTRUCTIONS FOR USE OF PRISM LEVEL NOTEBOOK 9-940 WHEN USED FOR PRIMARY OR PRECISE LEVELING RECORD.

Fill in the blanks on the flyleaf the first day the book is used.

Fill in the blanks at the head of each page each day, on precise work, indicating bench marks run between by their letters or numbers.

Each horizontal space between two red lines is for a single set-up of the level.

The notes for each section of line on precise work must be complete in themselves and commence on a new page. Every primary line record must begin on a new page, and the initial bench mark must be fully described.

The columns being counted from the left, each is used as follows:

Column 1 is for the readings on the rod in yards for the three threads, each set of readings to occupy a separate space between red lines, the first recorded reading being for the wire giving the smallest value. The color letter is to be placed beside the first and last readings. The recorder should notice whether the color as recorded corresponds with the unit called out by the levelman. Each day the levelman should verify the comparison and, if a discrepancy exists, rerun the section.

Column 2 is for the thread intervals for the thread readings in column 1, the upper ones being the difference between the lowest readings and the middle ones, the lower being the difference between the middle and the greatest readings of each set. (See also paragraph following.)

Column 3 is for the sums of the three-wire readings of each space in column 1, between the horizontally ruled red lines, these sums being equal to the mean in feet of the three readings on rod.

Column 4, with the exception of the last line, is not intended for use in precise leveling, but can be used to compute approximate elevations, being filled out only at bench marks. On primary work the first entry on the page at the left of the words "Elevation brought forward from page —" should be the elevation from a previous page, or from another book. In the latter case, give book number and page, and always carefully verify the copying. The second entry, below the red line and above the short black line, is the height of the instrument as found by adding the first entry in column 3 to the first elevation in column 4. The third entry in column 4 is the elevation computed by subtracting the first fore sight from the H. I. In each case the H. I. will always be above a short black line and the elevation always just above a red line.

The records in columns 3 and 5 should be placed on line with the H. I.

Columns 5, 6, and 7 are for fore-sight readings, corresponding with 3, 2, and 1 for back-sight readings.

Column 8 is for the record of temperature and time.

Column 9 is for the correction of curvature and refraction for unequal sights and need not be filled out in the field.

Column 10 is for extra fore sight at points which are not turning points, also for their sum.

Column 11 is for description of bench marks, for elevations from extra fore sights, for transcripts of bench-mark elevations, and for general remarks or explanation.

In columns 2 and 6 write next above the red lines the continuous sums of the rod intervals for the section. The mean of the last pair of continuous sums in columns 2 and 6, multiplied by 1,000, will be equal to the distance in feet for the page; its equivalent in miles and tenths can be obtained from the table in the back of the book. The total mileage from the beginning of the section on precise work and of the line on primary work must be given at the bottom of each right-hand page.

A sample page from a field book follows:

Date: *July 6, 1910, A. B. C*, Lev., *D. E. F.* Rec. Line from *B. M. 53*, via *Forward* to *B. M. 54.*

Back sight. Thr. rdg.	Back sight. Thr. int.	Back sight. Sum. t. r.	H. I. feet. / Elev. feet.	Fore sight. Sum. t. r.	Fore sight. Thr. int.	Fore sight. Thr. rdg.	Temp.	Dif. cor. C. & R.	Extra fore sight.	Descriptions, elevations by extra fore sight, and transcript of other elevations.
			1150.634	Elevation brought forward from page 74.						
R 1.323 *1.569* *1.814*	*.246* *.245* *.491*	*4.706*		*3.807*	*.242* *.242*	*R 1.027* *1.269* *R 1.511*	*25A*			*10.20 a. m. Strong wind.*
R 1.018 *1.259* *R 1.501*	*.241* *.242* *.974*	*3.778*		*4.413*	*.240* *.240* *.964*	*R 1.231* *1.471* *R 1.711*	*25B*			
R 1.382 *1.624* *R 1.865*	*.242* *.241* *1.457*	*4.871*		*4.797*	*.241* *.241* *1.446*	*R 1.358* *1.599* *R 1.840*	*25A*		*B*	
R 1.382 *1.586* *R 1.790*	*.204* *.204* *1.865*	*4.758*							*R 1.840* *1.888* *R 1.936* *5.664*	*18.113* *18.681* *−0.568=53 to road crossing, top of rail.*

				6.174	.209 .209 1.864	R 1.849 2.058 G 2.267	24B			*No. 54=railroad spike driven in south side of telegraph pole 0.35 mile east of mile pole 303.*
										1150.634=No. 53. 10.35 a. m. *−1.078* *1149.556=No. 54.*
			+18.113 −19.191							
Sums for page.			Elev.—Elev.	Sums for page.			Mean.	Sum.		
6.038 3 18.114 −1 18.113	1.865	18.113	B.S—F.S. −1.078	19.191	1.864	6.397 3 19.101	25		(Col. 2 + Col. 6) ÷ .002 = $\frac{3.729}{1,864}$ feet = 0.35 miles; 3 total miles. 3 C (Col. 2 − Col. 6) = ... cor. for level and collimation in feet.	

COMPUTATION AND ADJUSTMENT OF LEVEL CIRCUITS.

The computation of each line is first checked to be sure that the proper corrections have been made for errors of rods, including those due to changes in temperature, and for errors due to unbalanced sights as affected by curvature and refraction. Corrections are then made for systematic errors for which the law may be known, such as that necessary to take account of the fact that water levels along a meridian at different altitudes are not parallel curves, except at the equator and at the poles. This correction depends on meridional distance, latitude, and altitude, and is called the orthometric correction; it may be found from the following approximate formula:

$$C=\frac{h_m\,(\phi_2-\phi_1)\sin\,(\phi_2+\phi_1)}{660{,}000}$$

in which

C= correction in feet.

h_m= mean height of line in feet.

ϕ_1 and ϕ_2= are the latitudes of the south and north ends of section, respectively.

$(\phi_2-\phi_1)$= difference of latitude in minutes of arc.

In applying the formula the lines must be divided into sections of not over 100 miles each, and a division should be made where the general direction changes materially. The corrections thus found are applied to the several sections so as to lower the elevations at successive division points going northward. Although orthometric corrections at times lead to apparently absurd results, such as giving a lower elevation for the north end of a large lake having no outlet than for the south end, yet in order to insure agreement between different lines and to obtain results of the greatest theoretical accuracy, they must be applied when appreciable.

After all the foregoing corrections are made to the original results, the remaining closure errors are those which are to be removed by adjustment.

Precise.—Weights are first assigned for each class of levels, and observation equations are formed and solved by "least squares." In this manner every line helps to establish the elevation for each junction point. When all the junction points are fixed the corrections are distributed over the lines in proportion to distance.

Primary.—The Geological Survey in adjusting primary levels has adopted a method which may be described as follows:

All adjustments are to be made in the bench-mark description book 9–916, in which abstracts from the field books, which include the description and elevation of each point as determined by the levelman, are written by him in regular order for each line as run.

All the level lines associated with one another should be considered at one time, and in order to better comprehend their arrangement they should first be platted on the office progress maps as accurately as possible and from these tracings should be made on paper, to be used in the adjustment and later filed with the description book as part of the record. The plat should show the approximate relation of all the lines, including the precise or previously adjusted lines forming the base of the system, and the work of different grades or different men should be represented by differently colored inks or pencils or in some other manner, a suitable explanatory legend being attached. The names of a sufficient number of towns should be given to identify the location readily, and beside each line reference should be made to the page in the description book where the bench-mark elevations for that line are given. On each line a $>$ is to be placed to show the direction in which it was run. For small areas the diagram of routes prepared by the levelman in the description book will probably answer in place of the tracing.

The field notes should be examined to see whether the work was in accordance with the instructions; whether fore and back sights were equalized, rod readings properly summed, balances checked, and elevations properly copied from page to page. The entries in the description book should be systematically checked to see that all elevations, including those at starting, junction, and closing points, and all breaks and second runnings are properly copied. Where two runnings of equal weight are made over one course the mean result should be accepted for adjustment and written in red ink with the appropriate statement in the "unadjusted elevation" column, the divergence being given in the margin.

At each junction point on the diagram should be written the difference between the recorded elevation by some one of the lines and those recorded in the description book for each of the other lines for the same bench mark, with an arrow alongside and plus or minus signs added to indicate that the elevations as brought by these lines

are greater or less than the assumed one. Also, as an additional aid in the adjustment, the closure error for each circuit should be written in the center of its plat on the diagram, each amount and sign being computed in counter-clockwise order. Next ascertain by inspection of the diagram which of the unknown junction points may be determined with the greatest apparent accuracy or by the greatest number of independent lines. From two or more lines connecting this point with the points of known elevation obtain two or more possible corrections to the assumed elevation. Estimate and record relative weights for these corrections, the weights to be based on length of lines (usually in inverse proportion to their length), class of leveling instrument used, number of times leveled, and in rare cases on relative standing of observers if two are involved. Weights should not be influenced greatly by closure errors. Where corrections from different sources have a line in common, the length of this line should be doubled in fixing the weights of each.

From the weights adopted compute the weighted mean correction to the assumed elevation of the new point as follows: Multiply the correction computed for each of the independent lines from known points in turn by the weight of the line from which each was determined; divide the algebraic sum of these products by the sum of the weights. The quotient is the correction to apply algebraically to the assumed elevation; it should be written in the diagram at the proper junction point in a small loop or rectangle with the letters "Cor." and the plus or minus sign. In complicated nets it may be necessary to assign a preliminary correction to a junction point in order to carry a correction from it to some other point; after fixing the correction for the second point from its various lines a final correction is determined and substituted for the preliminary value of the first point.

In this manner weighted values are found for each junction point in turn, and between the points thus fixed corrections are distributed in proportion to the distance. A line or point once thus adjusted should not be readjusted unless readjustment is required by new field data.

Figure 2 is given as an illustration of the method of adjusting a level net. By inspection of the diagram, junction point E appears

to be the most favorably situated for adjustment first. The line run from A via B and E to D closed at D 0.250 foot low; from H via I to E 0.500 foot low. The elevation brought from source A is at first assumed for the adjustment as having a correction of 0. The corrections brought from sources D and H will then be +0.250 and −0.500, respectively. The distances to be used in assigning weights are taken as A via B to E$=\frac{10}{2}+8=13$ miles (A to B, being a double-run line, must be given double weight, which is done by dividing the length of the line by 2); D to E, 5 miles; H via I to E, 10 miles.

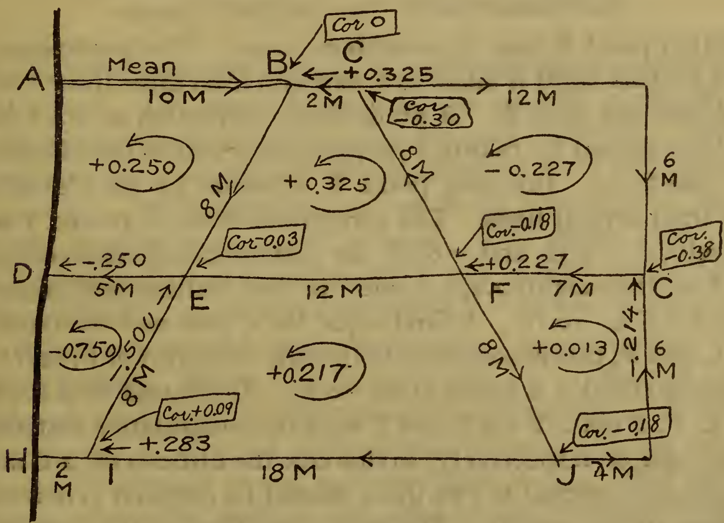

FIGURE 2.

The weights to be assigned should be in inverse proportions to the length of the lines, or nearly so. To determine the weights, divide any convenient number—as 13 in this example—by the computation distances 13, 5, and 10 each in turn, obtaining 1, 2.6, and 1.3 for the weights of the respective lines. These weights are each to be multiplied by the corresponding assumed corrections 0, +0.25, and −0.50, giving products of 0, +0.65, and −0.65. Divide the algebraic sum of these products by the sum of the weights (4.9), the quotient will be the weighted correction; this is 0 for the point in question, but as there is another line to this point which has not

been considered this correction must be accepted as preliminary only. The foregoing data may, if desired, be assembled in tabular form, thus:

From point—	Miles.	Weight.	Correction.	Weight × correction.
A....	13	1.0	0.000	0.00
D....	5	2.6	+ .250	+ .65
H....	10	1.3	− .500	− .65
Sum .	....	4.9	....	.00

Junction point B may be considered next. The preliminary correction for this point is taken as 0, as found from three lines, two lines from A and one from E. A preliminary correction of +0.1 foot for I can be obtained by taking a proportionate part of the closure error at E (one-fifth). Junction point F depends for its elevation on values from several lines. The corrections from E, B, and I are, respectively, 0, −0.32, and −0.18; the corresponding distances are 12, 10, and 26; the weights 2.2, 2.6, and 1.0; the resulting preliminary correction for F is −0.18. A final value for E may now be found from B, D, I, and F; this is necessary to include the effect of F, and by the foregoing method it is found to be −0.03. G is found from lines from B via C, F, F via C, F via J, and I, with the computation distances 38, 7, 44, 28, and 38, respectively; in this case the distances C to G and J to G, which are common to two lines, should be doubled in order not to give them undue weight. The final corrections to the assumed elevations are now found in a similar manner, the computations for B, I, and F being repeated to secure the effect of the additional lines, and are as follows: B, 0.00; C, −0.30; E, −0.03; F, −0.18; G, −0.38; I, +0.09; and J, −0.18, two places of decimals only being used for junction points. Each of these corrections is placed in a rectangle on the diagram near the point to which it belongs.

After the corrections for the junction points are fixed, corrections proportioned to the distance are found for intermediate points along the several lines.

Lines on which the closure error is much over the permissible limit must be omitted in adjustment; they may be tied in afterwards, but

in publishing the results a statement must be made cautioning engineers against dependence on them. If gross errors are evident, the results must not be published until the lines are rerun.

For long north-south lines n high altitudes, the orthometric correction should be applied (p. 70).

The computer should report to the division geographer in writing any failure on the part of the levelman to comply with instructions; he should also report all circuit-closure errors in excess of the allowable limit (pp. 57, 59, and 64).

ADJUSTMENT OF INSTRUMENTS.

The object glasses and eyepieces of all instruments must be properly focused. The cross wires projected against a distant object should appear immovable when the eye only is moved. Before the adjustments are commenced the instruments must be firmly set up and leveled. An instrument may at times appear to be out of adjustment because some part is loose. The object glass may be partly unscrewed or an adjusting screw may be only partly tightened. Level bubbles or cross wires occasionally become loosened; therefore, before commencing the adjustment of an instrument look out for such defects. When it is thought that an adjustment has been completed, always test it before using the instrument. All adjusting screws should be screwed tight enough to hold, yet not so tight as to injure the threads or put a severe strain on any other part. Adjustments should be made in the order given, for many adjustments depend on the accuracy of others previously made; sometimes two or more adjustments must be made simultaneously, as a change in any one affects the others.

TELESCOPIC ALIDADE.

But two adjustments are required for the telescopic alidade—for level and for collimation. These should be tested daily.

Level.—Clamp the telescope, bring the bubble to the center of the tube with the tangent screw, lift up the level carefully, reverse, and replace it on the telescope. If the bubble runs away from the center bring it halfway back by means of the tangent screw and the other

half by the adjusting screw under the end of the level tube. Repeat this operation till the bubble stays in the center after reversal.

Collimation.—Point the telescope on a small but well-defined object about half a mile distant, and while watching this through the telescope revolve the telescope 180° in its supporting sleeve. If the intersection of the cross wires remains centered on the object the adjustment is perfect; if not, change the cross wires for half the error and repeat the operation until they stay on the point selected. A slight shift in the position of the cross-wire ring should be made if necessary to make the vertical wire parallel to the vertical corner of a building or a plumb line.

Ruler.—So long as but a single alidade and but one edge of the ruler are used it makes no difference in the results whether the edge of the ruler is parallel to the line of sight or not.

Y LEVEL.

All instrumental errors of the Y level can be eliminated by exactly equalizing fore and back sights, but as this is seldom possible the line of collimation and the level should be kept as nearly in adjustment as practicable.

Level.—Having the instrument carefully leveled, loosen the clips, lift the telescope out of the Ys, reverse it end for end, and replace it in the Ys; if the level bubble has moved away from the center bring it halfway back by means of the adjusting screws at one end of the level tube and the other half by the lower leveling screws. Repeat this operation until the adjustment is perfect. With the bubble in the center, rock the telescope back and forth in the Ys about 25° around its axis; if the bubble moves away from the center bring it back with the side adjusting screws.

Collimation.—Having the instrument carefully leveled, note a small object about 300 feet distant that one end of a horizontal cross wire touches, turn the instrument on its vertical axis a few degrees, and note whether the other end of the cross wire cuts the point; if it does and the Ys are not badly out of adjustment the wire is horizontal. With the clips up, focus on a small object 300 or 400 feet distant; watch this through the telescope while revolving it 180° in the Ys; if the intersection of the cross wires moves away from the point bring it

halfway back by means of the cross-wire adjusting screws; repeat the test and adjustment until there is no movement of the cross wires away from the point.

Object-glass slide.—It is seldom necessary to adjust the object-glass slide, as it is usually fixed by the maker, but when required make the collimation adjustment as above described; then an error in the adjustment of the slide will appear as an error of collimation when tested on a near-by point, say 50 feet distant. To correct the error remove the ring near the middle of the telescope and with a screw driver turn the screws found underneath so as to bring the cross-wire intersection halfway back to the near-by point selected

Eyepiece slide.—The adjustment of the eyepiece tube so that the cross wires will appear in the center of the field, though not essential to the accuracy of the work, may be effected by means of the screws underneath the ring just back of the cross-wire screws. Loosen one and tighten the opposite one of these screws with a screw driver until the wires appear centered.

Ys.—After each of the foregoing adjustments have been made, the adjustment of the Ys is made by turning the telescope and level 180° on its vertical axis; if the level bubble, which was at first in the center, moves away from it, bring it halfway back by changing the large nuts under one Y.

"*Peg method.*"—In the ordinary Y level adjustment it is assumed that the two rings on the telescope tube which rest in the Ys are circular and exactly equal by construction.

The level and line of collimation can be made parallel independently of the rings and Ys by the "peg method" described under the heading "Prism level" (p. 60).

LOCKE LEVEL.

The adjustment of the hand level or Locke level is most easily tested by sighting along a horizontal line determined by a Y level or alidade, but when no such line is available, a modified form of the "peg method" must be used. Hold the level on a fixed point and sight a second point 300 or 400 feet distant which appears by the level being tested to have the same elevation as this point. Take the level to the second point and with the bubble centered over the cross

wire sight the first point; if it appears to be on the horizontal line the level is in adjustment; if not, correct for one-half the difference by turning the small screw at one end of the level box.

ROD LEVEL.

The leveling or stadia rod to which levels are attached should be carefully plumbed with string and plumb bob. The level bubbles should then be brought to the centers by means of the proper adjusting screws.

TRANSIT.

Plate levels.—After leveling carefully revolve the instrument 180° on its vertical axis. Bring each level bubble halfway back to the center of the tube by means of the screw at one end.

Collimation.—Level carefully, sight on a point about 500 feet distant, raise or lower the telescope slightly, and note whether the vertical wire remains on the point; if not, loosen the capstan-headed screw and turn the cross-wire ring till the vertical wire will remain on the point when the telescope is raised or lowered. Clamp the instrument, set the vertical wire so that it cuts the point selected, transit the telescope by revolving it 180° on its horizontal axis, and select a second point 500 feet distant in the opposite direction from the first. Unclamp the upper plate, turn the transit 180° on the vertical axis, set it on the point first selected, and again clamp the plate. Transit the telescope and if the vertical cross wire exactly bisects the second point the adjustment is perfect; if it does not, bring it one-quarter of the way back to the second point by turning the two capstan-headed screws on the sides of the telescope.

Standards.—Set up the transit near a tall building or other high object; after leveling carefully point the telescope so that the vertical wire intersects a definite point about 60° above the horizontal, depress the telescope, and select a second point near the ground. Unclamp the upper plate, revolve the telescope and plate 180° on the vertical axis, clamp the plate with the vertical wire again cutting the upper point, and depress the telescope; if the cross wire intersects the lower point the standards are in adjustment; if it does not, correct for one-half the error by the screw underneath one end of the telescope axis.

Object-glass slide.—If an adjustment for the telescope object-glass slide is possible, it is made as follows: First make the collimation adjustment for a point about 300 feet distant, then focus on a point 1,000 feet or more distant and again on a point only 10 or 15 feet away, transit the telescope, unclamp the plate, turn it 180° on the vertical axis, and reclamp. If the cross wire still cuts the distant and near points the slide is in perfect adjustment, but if it does not, correct half the error by means of the side screws which hold the slide ring in place. Next repeat the regular collimation adjustment and again test for the slide error; repeat both adjustments until no errors appear.

Eyepiece tube.—The eyepiece may be put into position over the cross wires by turning the screws which hold the eyepiece ring until the cross wires appear in the center of the field; an exact centering is not required.

Telescope level.—If there is a level attached to the telescope it may be adjusted by the "peg method" after all the other adjustments are made, as follows: Level the transit and bring the bubble to the center of the tube under the telescope. Take a reading on a leveling rod or pole 300 or 400 feet distant, which is held on a stake set firmly in the ground. Revolve the transit 180° on the vertical axis and after again bringing the bubble to the center set a second stake at the same distance as the first and at such an elevation that the rod or pole reading is the same as on the first stake. The tops of the two stakes will then be at the same elevation. Move the transit 25 or 50 feet back of one stake and on a line with the other. Make the telescope as nearly horizontal as possible by means of the attached level, clamp it, and then take a reading on the rod held on the near stake and another reading on the distant stake. If the two readings agree the telescope is horizontal; if they do not agree turn the tangent screw so as to bring the cross wire while set on the distant rod nearly to an agreement; repeat the operation till an agreement is reached. The telescope is then level and the adjusting nuts at the end of the level tube should be turned till the bubble is brought to the center.

Vertical circle or arc.—The screws holding the vernier for the vertical arc should now be loosened and the vernier moved until the reading is 0° while the telescope is still level.

THEODOLITE.

Striding level.—Place the level in the proper position on the telescope axis. Level carefully with the horizontal plates clamped and rock the level slowly back and forth till the foot pieces strike. If the bubble leaves the center, bring it back by means of the side adjusting screws near one end of the tube.

Reverse the level and bring the bubble halfway back to the center by raising or lowering one end of the tube with the screw at that end, and the other half with the leveling screws. Repeat these operations till the adjustment is perfect.

Standards.—After the striding level is in adjustment with the lower horizontal circle clamped, level the instrument in two positions at 90° from each other. Turn on the vertical axis 180° from one position; if the bubble runs away from the center bring it halfway back by loosening one of the large capstan-headed screws underneath the standards and tightening the other. Test the adjustment and repeat it if necessary.

Plate levels.—Level instrument with the striding level only, then bring the bubbles of the plate levels to the center of their tubes by means of the end adjusting screws; or the method described for adjusting the transit plate levels may be used for the theodolite also.

Micrometers.—Each micrometer consists of three concentric tubes; the upper and lower ones slide in the central one. The lower tube, which holds the object lens when in proper position, is clamped to the middle one by means of the capstan-headed screw in the lower part of the]-shaped support. These two tubes may be moved together or the lower one moved alone by loosening the proper screws. The upper tube contains the eyepiece lenses and is held in place by friction only.

Focus the eyepiece on the two parallel movable threads and do not change it afterwards. With the eye in position for setting the micrometer, tighten one and loosen the other of the two screws that hold the]-shaped microscope support to the main frame of the theodolite, until the figures and graduations on the plate appear to be in the center of the field.

Clamp the plate and by turning the micrometer screw set the two movable threads over a long graduation. Examine carefully to see

whether they appear exactly parallel to it. If they are not parallel, loosen the two capstan-headed screws which clamp the micrometer tube and twist the tube until the threads and mark appear parallel. Clamp the side screws lightly.

Set the movable cross wires on a division to the apparent left of the field of view as for a regular angle reading; read the micrometer head and record the reading. Turn the graduated head about five turns, stopping when the threads are set on the next 10′ division to the right; read and record. Repeat this operation several times. If the mean of the left-hand readings is the same as the mean of the right-hand readings, or within one division of it, the adjustment may be accepted as satisfactory. An actual count of full revolutions should be made at least once; otherwise the adjustment might wrongly be thought perfect for 4½ or 5½ revolutions.

When the space covered by the two parallel micrometer threads, moved by exactly five revolutions of the micrometer screw, appears to be longer than one 10′ space on the graduated circle, to bring it into adjustment make the distance between the micrometer box and graduated plate longer by raising the middle part of the tube; but when the space is shorter than a 10′ space make that distance shorter also—that is, consider as connected or dependent the length of the thread space covered by an even five revolutions of the micrometer screw and the distance between the micrometer box and the graduated plate. When the former is longer than it should be, the latter should be made longer, if an adjustment is desired, and vice versa.

To make the adjustment, loosen the small capstan-headed screws which clamp the microscope tube; then, if the thread space is long, twist the middle part of the tube (including the micrometer box) back and forth and at the same time pull it upward, thus lengthening the distance to the graduated plate. When by estimation it has moved far enough, which can be roughly determined by the amount of blurring that results from the lower lens being thrown out of focus, clamp the upper capstan-headed screw. The lower part of the microscope tube holding the objective lens must now be twisted and gently pushed downward till the graduations again appear in focus. If the movable threads and graduations are not then parallel, the upper screw must be again loosened and the tube turned far enough to make

them parallel, after which both screws must be tightened. Test the adjustment by again measuring a $10'$ space with the micrometer. If it is still out of adjustment, repeat these operations till it is satisfactory. When the adjustment has been completed, a scratch may be made on the tube below each support and used as a guide in future adjustments.

The opposite micrometers may be placed 180° apart by setting one at a reading of $0^\circ\ 0'\ 0''$, with the comb scale exactly centered. Then center the comb scale of the other micrometer over the 180° mark by means of the capstan-headed screw at the left-hand end of its box. Bring the micrometer threads over the 180° mark also; then, while holding the screw firmly in place, turn the graduated ring till it reads zero.

When setting the micrometer wires on a graduation, it is very important that they be moving toward the right when the turning of the screw is stopped. Should they be moved the least bit too far to the right, turn back not less than half a revolution of the screw and then bring them forward again. In general, when a setting is made by means of a screw working against a spring, the spring should always be undergoing compression when the motion stops.

Cross wires.—The vertical wire should be truly vertical; otherwise an exact adjustment of the cross wires is not essential.

After the striding level has been adjusted and the horizontal axis of the telescope carefully leveled, sight a distant point, raise and lower the telescope through an angle of 5° or 10°, and note whether the cross wires follow the point. If not, loosen the cross-wire ring and twist slightly; repeat the adjustment if necessary.

Hold the striding level on the telescope parallel to the optical axis and, with the bubble in the center of the tube, set the intersection of the cross wires on a distant point and clamp both plates; lift the telescope out of its supports and turn 180° around its optical axis; set it again on the selected point. If the striding level when placed on top of the telescope is horizontal, the adjustment is complete. If not, shift the cross wires in either direction by means of the capstan-headed screws for one-half the apparent error. Repeat the test till the error is nearly all eliminated. Finally readjust the vertical wire, if necessary.